

For Reference

NOT TO BE TAKEN FROM THIS ROOM

Ex LIBRIS
UNIVERSITATIS
ALBERTAENSIS





Digitized by the Internet Archive
in 2019 with funding from
University of Alberta Libraries

<https://archive.org/details/Smith1972>

THE UNIVERSITY OF ALBERTA

THAW CONSOLIDATION TESTS ON REMOULDED CLAYS

by



LAURENCE BRUCE SMITH

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES AND RESEARCH
IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE
OF MASTER OF SCIENCE

DEPARTMENT OF CIVIL ENGINEERING

EDMONTON, ALBERTA

SPRING, 1972

Thesis
1972
134

THE UNIVERSITY OF ALBERTA

FACULTY OF GRADUATE STUDIES AND RESEARCH

The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies and Research, for acceptance, a thesis entitled THAW CONSOLIDATION TESTS ON REMOULDED CLAYS submitted by Laurence Bruce Smith in partial fulfilment of the requirements for the degree of Master of Science

ABSTRACT

A general solution to the problem of one-dimensional thaw consolidation has been formulated by Morgenstern and Nixon (1971). In order to assess the validity of the theory it was necessary to develop a special oedometer (Permode) which could impose the necessary thermal and stress boundary conditions for one dimensional thaw consolidation.

The permode permits the measurement of settlements, temperatures at various depths on the side of the sample, and excess pore pressures at the base of the sample during thaw-consolidation.

Controlled thaw consolidation tests were carried out on three types of remoulded clays. The resulting data showed that the excess pore pressures and the degree of consolidation in a thawing soil depend primarily on the thaw consolidation ratio. The results obtained demonstrate that the theory adequately represents the soil behavior provided that the soil exhibits a linear relationship between void ratio and effective stress.

ACKNOWLEDGEMENTS

I wish to express my sincere appreciation to the following:

Professor N. R. Morgenstern for suggesting the topic and for his continued guidance and enthusiasm throughout the project.

Professor J. F. Vaneldik who designed and built the temperature controllers used in the experiments.

Mr. J. F. Nixon for his comradeship during the past eighteen months, and for assisting in the testing program, aiding in the analysis and interpretation of the data and for clarifying many theoretical aspects.

Mr. P. K. Chattopadhyay who provided an endless supply of helpful suggestions and encouragement.

Mr. A. Muir who built the permode and suggested several improvements to the design.

Mrs. J. F. Nixon who did a flawless job of typing the thesis.

My wife, Mary Lou, who kept supper warm on innumerable occasions.

TABLE OF CONTENTS

TITLE PAGE	Page i
APPROVAL SHEET	ii
ABSTRACT	iii
ACKNOWLEDGEMENTS	iv
TABLE OF CONTENTS	v
LIST OF TABLES	vii
LIST OF FIGURES	viii
CHAPTER I - INTRODUCTION	1
1.1 General introduction and review of past research	1
1.2 Purpose of thesis	6
CHAPTER II - THEORETICAL	7
2.1 Summary of the Morgenstern-Nixon Thaw Consolidation Theory	7
2.2 Theoretical predictions for the oedometer case	14
CHAPTER III - DESIGN AND DESCRIPTION OF THE PERMODE	19
3.1 Preliminary testing	19
3.2 Pore pressure measurement during consolidation	27
3.3 Description of the permode	37
CHAPTER IV - THAW CONSOLIDATION TESTS	42
4.1 Test procedure - Thaw consolidation	42
4.2 Calculations	44
4.3 Test results	54
CHAPTER V - DISCUSSION AND CONCLUSIONS	66

5.1	Validity of the thaw consolidation theory	66
5.2	Sources of error	76
5.3	Recommendations for improvements to apparatus	89
5.4	Conclusions	92

LIST OF REFERENCES	95
--------------------	----

APPENDIX A	Summary of soil properties	103
APPENDIX B	Specifications of major components of temperature control devices	108
APPENDIX C	Summary of data test series 5-4	111
APPENDIX D	Summary of data test series 5-5	121
APPENDIX E	Summary of data test series 5-6	126
APPENDIX F	Summary of data test series 5-7	140
APPENDIX G	Summary of data test series 5-8	146
APPENDIX H	Summary of data test series 5-9	151

LIST OF TABLES

	Page
TABLE 3.1 Consolidation results Athabasca Clay not subjected to freezing	21
TABLE 3.2 Consolidation results Athabasca Clay subjected to one freeze cycle	22
TABLE 4.1 Summary of results test series 5-4 (Example)	47
TABLE 5.1 Summary of thermal calculations - Athabasca Clay	85

LIST OF FIGURES

	Page
Fig. (2.1) One-dimensional thaw consolidation	8
Fig. (2.2) Excess pore pressures	15
Fig. (2.3) Maximum base excess pore pressure versus log R	16
Fig. (2.4) Degree of consolidation during thaw versus log R	16
Fig. (2.5) Post thaw excess pore pressure dissipation at base of sample	17
Fig. (2.6) Theoretical settlement curves for post thaw consolidation	18
Fig. (3.1) Void ratio versus log effective stress - Athabasca clay	23
Fig. (3.2) Void ratio versus log permeability - Athabasca clay	25
Fig. (3.3) Diagram of the permode	29
Fig. (3.4) Base pore pressure dissipation curve for standard consolidation - no membrane	32
Fig. (3.5) Base pore pressure dissipation curve for standard consolidation - no membrane	32
Fig. (3.6) Base pore pressure dissipation curve for standard consolidation test - with membrane	34
Fig. (3.7) Base pore pressure dissipation curve for standard consolidation test - with membrane	34
Fig. (3.8) Base pore pressure dissipation curve for standard	35

consolidation test - with membrane

Fig. (3.9)	Schematic diagram of test layout for thaw consolidation tests	38
Fig. (3.10)	Circuit diagram for the temperature controller	40
Fig. (4.1)	Thaw consolidation data test 5-4-3	46
Fig. (4.2)	Post thaw settlement and pore pressure dissipation curves test 5-4-3	50
Fig. (5.1)	Maximum pore pressure at base versus log R	68
Fig. (5.2)	Post thaw excess pore pressure dissipation at base	69
Fig. (5.3)	Degree of consolidation during thaw versus log R	71
Fig. (5.4)	Void ratio versus coefficient of consolidation	74
Fig. (5.5)	Normalized depth to thaw plane versus normalized square root of time	74
Fig. (5.6)	Variation of temperature at base of sample with square root of time	81
Fig. (5.7)	Temperature isochrones for test 5-4-4	81
Fig. (5.8)	Relationship between rates of thaw (α) determined theoretically and experimentally (Athabasca clay)	86
Fig. (5.9)	Improved permode design	90

CHAPTER I

INTRODUCTION

1.1 General Introduction and Review of Past Research

In recent years there has been a greatly accelerated industrial development in the Arctic regions of Canada and Alaska. This development has been accompanied by an increase in the number of heavy structures being constructed on permafrost. The need for quantitative design criteria for foundations placed on permafrost has therefore become urgent.

The basic information required in the design of any foundation is the strength and deformation properties of the subgrade soil. In general the strength and deformation of frozen soil is such as to present few foundation problems for the large percentage of structures where relatively light loads are involved.

For foundations which thaw the soil as well as impose a load on it the situation can be much more critical. For structures such as large reservoirs, some knowledge of the pore pressures developed as thaw progresses is necessary in order to economically design the sand drains and embankment slopes. A knowledge of the rate of settlement and the associated pore pressures is required in order to design foundations for hot oil and gas lines constructed over perma-frost terrain. In general, the foundation design for such structures requires a determination of the following:

- 1) The basic strength parameters for the thawed soil

ϕ' and c' ,

- 2) The rate of pore pressure development and dissipation as a function of the rate of thaw and the consolidation parameters for the soil,
- 3) The deformations associated with the thaw-consolidation process.

The basic strength parameters for a thawed soil can be determined by any of the test procedures presently employed in normal investigations of the mechanical properties of soils.

The correlation between the rate of thaw and the associated pore pressures is of fundamental importance to the design of foundations and in assessing the stability of slopes. Tsytoich et al (1965) was the first to establish experimentally that the rate of settlement of a thawing soil was directly proportional to the rate of thaw. Once thawing was stopped, Tsytoich indicates that the rate of settlement would be drastically reduced. Tsytoich recognized two important aspects of the thaw consolidation process. They are:

- 1) The pore pressures in the thawed zone dissipate according to the well known Terzaghi consolidation theory.
- 2) The pore pressure slightly above the thaw boundary remains constant provided the rate of thaw is proportional to the square root of time.

Tsytoich found that the pore pressures at the thaw boundary were some fraction of the applied load. That is:

$$U = (1 - m) P_0 \quad (1.1)$$

where U denotes the excess pore pressure at the thaw boundary,
 P_0 denotes the external applied load
 and m denotes a factor which is determined experimentally.

Tsytoovich provides values of m for different soils and formulates an equation for excess pore pressures using the above condition as a lower boundary. He was able to show that the excess pore pressures developed throughout the thawed portion of the soil depend not only on m but also on the ratio $\frac{\alpha}{2\sqrt{c_v}}$,

where α denotes a thermal constant indicating the rate of thaw,
 and c_v denotes the coefficient of consolidation for the soil.

It is difficult to assign a correct value to the coefficient m in design work and for this reason the pore pressures at the thaw boundary were usually assumed to be equal to the applied load. This leads to over conservative designs in many cases. Furthermore, the coefficient gives us no indication as to what is physically going on at the thaw boundary.

Zaretskii (1968) formulated a different boundary condition at the thaw boundary. He proposed that the change in water content of the soil as the thaw front moves a distance Δx is equal to the flow from the boundary in time Δt . That is if:

$$\Delta \bar{\omega} = S \frac{\Delta e}{G_s} \quad (1.2)$$

where $\Delta \bar{\omega}$ denotes the change in water content,

S denotes the degree of saturation ($= 1.0$),

Δe denotes the change in void ratio,

and G_s denotes the specific gravity of the soil grains.

Then,

$$\frac{\Delta e}{G_s} = \frac{k \frac{\delta u}{\delta x}}{\gamma_w \frac{\delta x}{\delta t}} \quad (1.3)$$

The boundary condition at the thaw line can then be derived as:

$$P_o - U(x,t) = \frac{G_s}{1+e} \frac{c_v \frac{\delta u}{\delta t} (x,t)}{\frac{\delta x}{\delta t}}$$

Zaretskii solves the classical consolidation equation subject to the above boundary condition. He is able to evaluate the coefficient m and therefore predict pore pressures and settlement. Zaretskii's assumption that the change in water content is equal to the volume of water which flows away from the thaw boundary is in error.

W. G. Brown and G. H. Johnston (1971) attempted to show that the depth of thaw and the corresponding settlement can be predicted using transient heat conduction theory. The depth of thaw can be predicted fairly well in this way, however, there is no attempt to relate the rate of thaw to the consolidation parameters for the soil and therefore it is impossible to predict pore pressures or their rate of dissipation.

A mathematical formulation for the thaw consolidation process has been derived by Morgenstern and Nixon (1971) using well known theories

of heat conduction and the classical Terzaghi consolidation theory. The approach is similar to that taken by Zaretskii but, there is a fundamental difference in the boundary condition employed at the thaw boundary. A summary of the theory is presented in Chapter II.

1.2 Purpose of Thesis

The purpose of this thesis was to ascertain the validity of the thaw consolidation theory as formulated by Morgenstern and Nixon (1971), by carrying out a series of temperature controlled one-dimensional thaw-consolidation tests on suitable remolded soil specimens.

In order to carry out such experiments it was necessary to design a specially adapted oedometer (Permode), which could simulate the required temperature and stress boundary conditions. The initial temperature must be accurately controlled in the apparatus until some desired step temperature and total pressures are applied. Both heat flow and consolidation must be one-dimensional once the thaw-consolidation test begins.

The permode was designed to meet the above requirements and permits the measurement of settlement, pore pressures at the base of the sample and temperatures on the side of the sample during the thaw-consolidation test.

Comparison of the results of such tests with the predictions obtained from the Morgenstern and Nixon (1971) thaw consolidation theory, provide a good indication as to the validity of the theory. Such tests also serve to further the present understanding of the freeze-thaw phenomena of porous media and open up new areas for future investigation.

CHAPTER II

THEORETICAL

2.1 Summary of the Morgenstern-Nixon Thaw Consolidation Theory

The mathematical formulation and solution to the problem of thaw consolidation which was derived by Morgenstern and Nixon (1971) is summarized here.

A one-dimensional configuration is considered where a step increase in temperature is imposed at the surface of a semi-infinite homogeneous mass of frozen soil. Refer to Fig. (2.1). The solution to the heat conduction problem is attributed to Neumann and is given by Carslaw and Jaeger (1947). The movement of the thaw plane is given by

$$X(t) = \alpha t^{1/2} \quad (2.1)$$

where X denotes the distance to the thaw plane

and t denotes time

α denotes a constant determined in the solution of the heat conduction problem.

$$\alpha = 2\lambda\sqrt{\kappa_2} \quad (2.2)$$

where κ_2 denotes the diffusivity of the thawed region

and λ denotes the root of a thermal equation which is a function of the initial temperature, the step temperature imposed, the conductivities of the thawed and frozen regions, the latent heat of fusion of the solid phase and the specific heat of the solid and liquid phases.

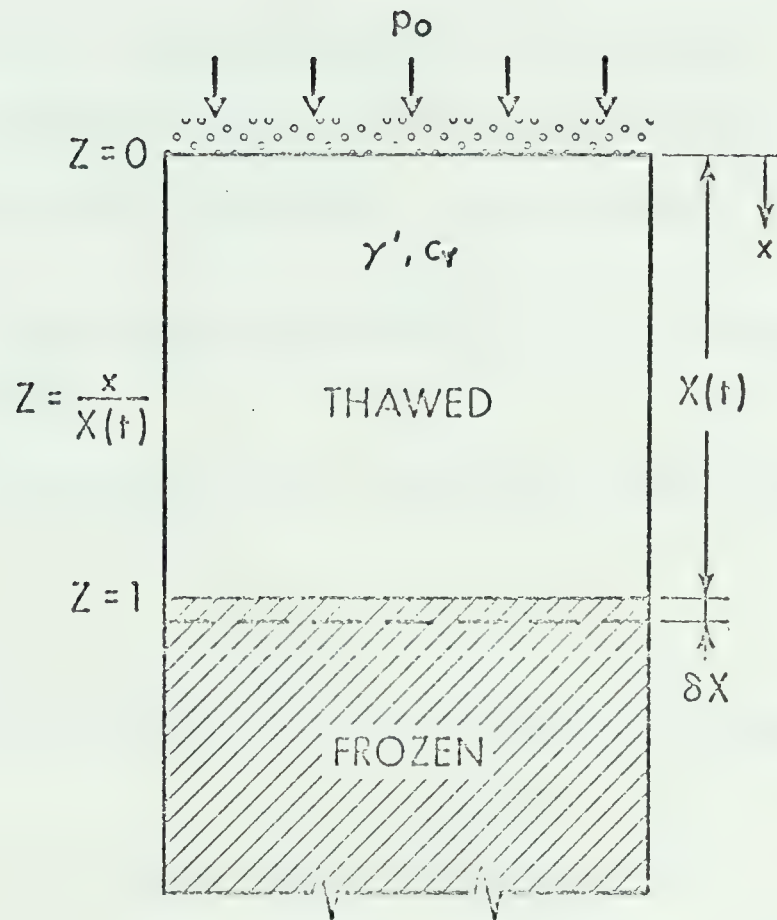


FIG. (2.1) ONE-DIMENSIONAL THAW CONSOLIDATION

In the thawed region, if it is assumed that the theory of consolidation is valid and that the total load does not vary with time then the governing equation is

$$c_v \frac{\partial^2 u}{\partial x^2} = \frac{\partial u}{\partial t} \quad (2.3)$$

where c_v denotes the coefficient of consolidation
 x denotes the depth from the ground surface
 and $u(x,t)$ denotes the excess pore pressure.

The foregoing equations must be solved, subject to the following boundary conditions.

(1) From equation (2.1) the initial condition is readily found to be:

$$t = 0; \quad x = 0$$

(2) Provided there is free drainage at the upper surface of the soil,

$$\text{for } t > 0; \quad u = 0; \quad x = 0.$$

(3) The boundary condition at the thaw line is derived as follows:
 The volume of pore fluid expelled from a small layer Δx as the thaw line advances in an increment of time Δt is given by Darcy's Law, so that

$$\Delta V = - \frac{Ak \frac{\partial u}{\partial x} (x,t)}{\gamma_w} \Delta t \quad (2.4)$$

where A denotes the cross-sectional area of a soil element
 k denotes the permeability of the soil
 $\frac{\partial u}{\partial x} (x,t)$ denotes the excess pore pressure gradient at the thaw interface

and γ_w denotes the unit weight of water.

The discharge ΔV must be equal to the change in volume of a layer of thickness ΔX . The volumetric strain is therefore,

$$\frac{\Delta V}{V} = \frac{\Delta V}{A \Delta X} = - \frac{k \frac{\partial u}{\partial x}(X, t)}{\gamma_w \frac{dX}{dt}} \quad (2.5)$$

But for a compressible soil,

$$\frac{\Delta V}{V} = -m_v \Delta \sigma' \quad (2.6)$$

where m_v denotes the coefficient of volume compressibility for the soil

and $\Delta \sigma'$ denotes the change in effective stress.

Substituting equation (2.6) into (2.5) will give,

$$\Delta \sigma' = \frac{c_v \cdot \frac{\partial u(X, t)}{\partial x}}{\frac{dX}{dt}} \quad (2.7)$$

where $c_v = \frac{k}{m_v \gamma_w}$, the coefficient of consolidation.

The total stress at $x = X$ is,

$$\sigma(X, t) = P_0 + \gamma X \quad (2.8)$$

where P_0 denotes the stress applied to the surface

and γ denotes the bulk density of the soil.

The pore pressure at $x = X$ is,

$$P_w(X, t) = u(X, t) + \gamma_w X \quad (2.9)$$

The effective stress is thus

$$\sigma'(X,t) = P_0 + \gamma'X - u(X,t) \quad (2.10)$$

where γ' denotes the submerged density of the soil.

Now,

$$\Delta\sigma' = \sigma'(X,t) - \sigma_0' \quad (2.11)$$

where σ_0' denotes the initial effective stress in the soil prior to any volume change on thawing.

It is safe to assume that in ice rich soils σ_0' will be very nearly zero. Furthermore, if σ_0' is substantial because of stress history effects, it would contribute to stability and make less severe the problems that arise upon thawing frozen soils. From equations (2.10) and (2.11) with $\sigma_0' = 0$, we find

$$\Delta\sigma' = P_0 + \gamma'X - u(X,t) \quad (2.12)$$

Substituting equation (2.12) into equation (2.7) gives the boundary condition at the thaw line as:

$$P_0 + \gamma'X - u(X,t) = \frac{c_v \frac{\partial u}{\partial X}(X,t)}{\frac{dX}{dt}} \quad (2.13)$$

An analytical solution to the governing equations subject to the above boundary conditions has been found by Morgenstern and Nixon to be:

$$u(X,t) = \frac{P_0}{\operatorname{erf}(R) + \frac{e^{-R^2}}{\sqrt{\pi} R}} \operatorname{erf}\left(\frac{X}{2\sqrt{c_v t}}\right) + \frac{\gamma' X}{1 + \frac{1}{2R^2}} \quad (2.14)$$

where $R = \frac{\alpha}{2\sqrt{c_v}}$, and is called the thaw consolidation ratio

and

$\operatorname{erf}\left(\frac{X}{2\sqrt{c_v t}}\right)$ is the error function.

Introducing the dimensionless variables

$z = \frac{X}{X(t)}$, the normalized depth of thaw

and

$W_r = \frac{\gamma' X(t)}{P_0}$, the ratio of the magnitudes of the applied load to the overburden pressure,

equation (2.14) becomes

$$u(z,t) = \frac{u(z,t)}{P_0 + \gamma' X} = \left(\frac{1}{1+W_r}\right) \frac{\operatorname{erf}(RZ)}{\operatorname{erf}(R) + \frac{e^{-R^2}}{\sqrt{\pi} R}} + \frac{Z}{\left(1 + \frac{1}{2R^2}\right) \left(1 + \frac{1}{W_r}\right)} \quad (2.15)$$

For a weightless material ($W_r = 0$), equation (2.15) becomes:

$$\frac{u(z,t)}{P_0} = \frac{\operatorname{erf}(RZ)}{\operatorname{erf}(R) + \frac{e^{-R^2}}{\sqrt{\pi} R}} \quad (2.16)$$

For a soil consolidating under its own weight ($W_r = \infty$)

equation (2.15) becomes:

$$\frac{u(z,t)}{\gamma'X} = \frac{Z}{\left(1 + \frac{1}{2R^2}\right)} \quad (2.17)$$

The average degree of consolidation can be defined as the ratio of the consolidation settlement that has occurred at time t to the total consolidation settlement that would occur if thawing were stopped at time t . Thus,

$$S = \frac{S_t}{S_{\max}} = \frac{\int_0^X \{P_0 + \gamma'x - u(z,t)\}dx}{\int_0^X \{P_0 + \gamma'x\}dx} \quad (2.18)$$

For the case of a weightless material ($W_r = 0$) the average degree of settlement will be given by

$$S = 1 - \frac{\text{erf}(R) + \frac{e^{-R^2} - 1}{\sqrt{\pi} R}}{\left(\text{erf}(R) + \frac{e^{-R^2}}{\sqrt{\pi} R}\right)} \quad (2.19)$$

and for $W_r = \infty$

$$S = 1 - \frac{1}{1 + \frac{1}{2R^2}} \quad (2.20)$$

Values of excess pore pressures and the average degree of settlement for various thaw-consolidation ratios have been determined and are provided in chart form in the original publication.

2.2 Theoretical Predictions for the Oedometer Case

It can be assumed that for the oedometer case $W_r = 0$. Thus the excess pore pressures will be given by equation (2.16). Values for excess pore pressure determined from this equation are presented in Fig.(2.2). The maximum theoretical pore pressures which will occur at the base of the sample in the oedometer for various R values can be derived from this figure and are presented in Fig.(2.3).

The average degree of consolidation for the oedometer case ($W_r = 0$) is given by equation (2.19). The degree of consolidation for various R values is presented in chart form in Fig.(2.4).

The pore pressures at the base of the sample in the post thaw phase of consolidation have been calculated by Nixon (unpublished) using a finite difference equation derived from the classical differential equation governing consolidation. The initial pore pressure distribution is taken as that which exists at the end of thaw and is therefore a function of the R value. The theoretical pore pressure dissipation curves evaluated for different thaw consolidation ratios is shown in Fig.(2.5). A theoretical post-thaw settlement curve can also be obtained for each R value and a set of such curves is shown in Fig.(2.6). This figure demonstrates that the theoretical settlement curve is almost insensitive to R for values of R less than 1.5. Furthermore it is apparent from this curve that if R is 1.5 or less, the time factor at fifty percent consolidation has a value of 0.28 as compared with a value of 0.197 which applies for a uniform initial pressure distribution.

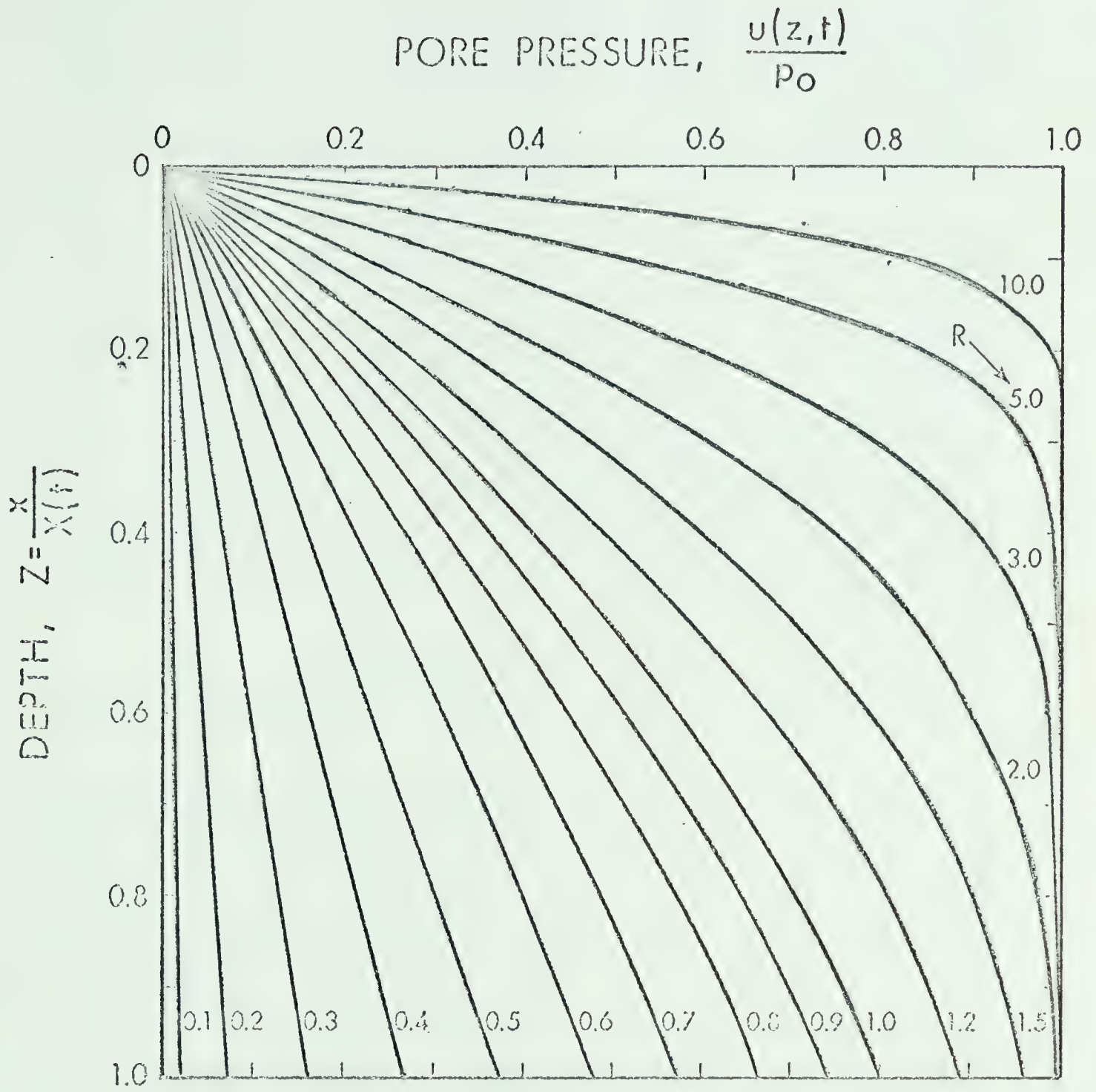


FIG. (2.2) EXCESS PORE PRESSURES, $w_f = 0$
(weightless material)

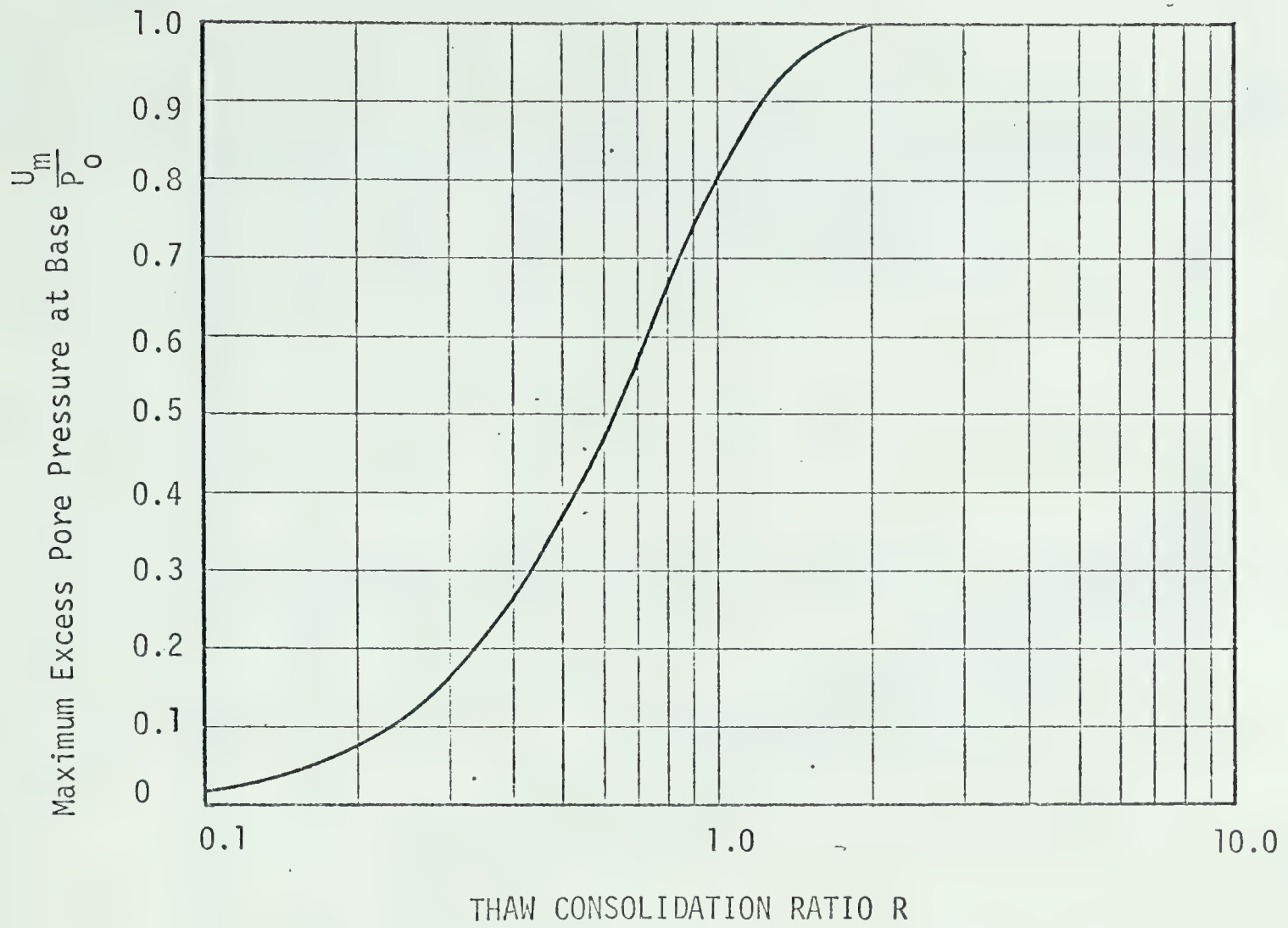


Fig. (2.3) Maximum Base Excess Pore Pressure Versus Log R

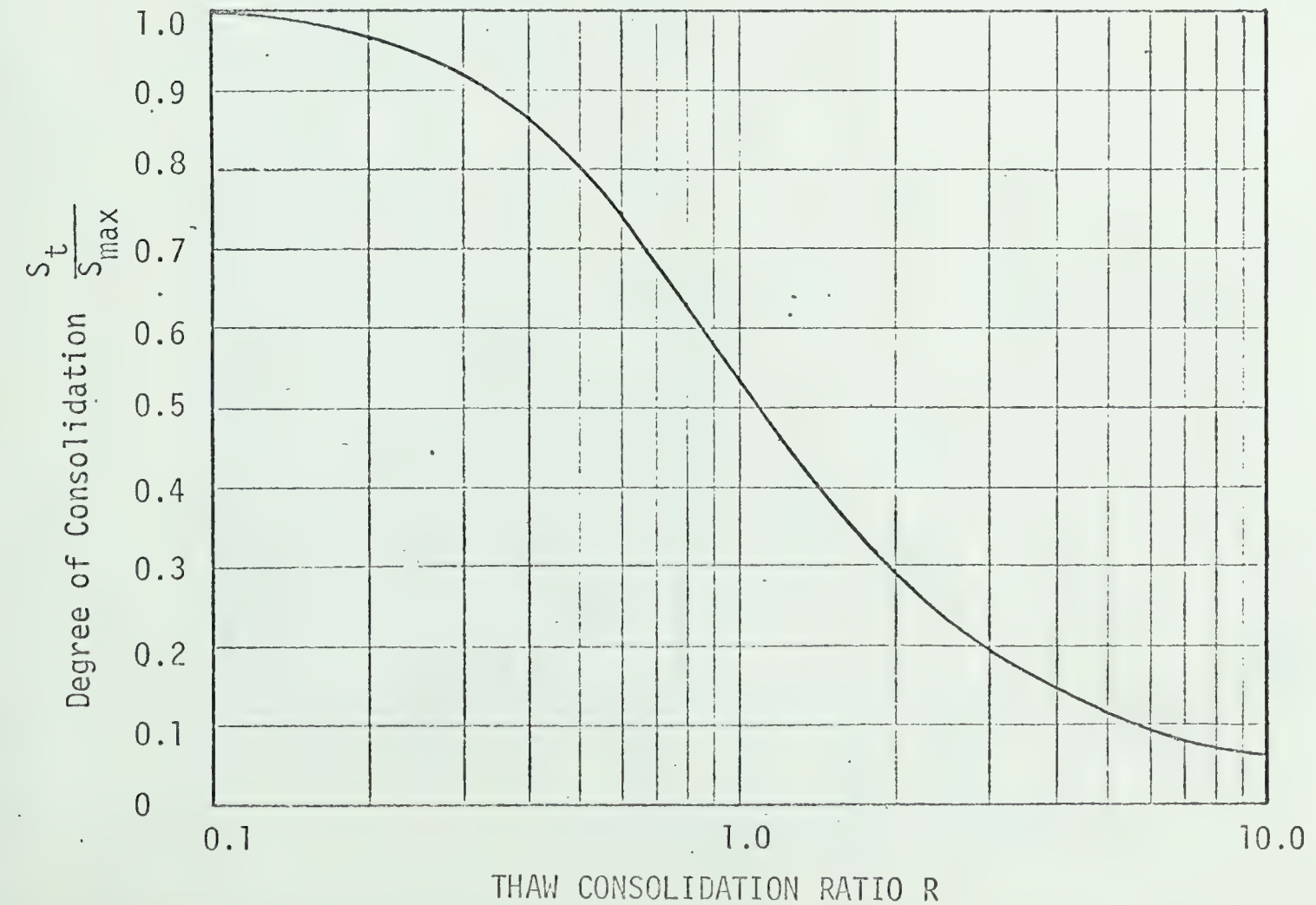


Fig. (2.4) Degree of Consolidation Versus Log R

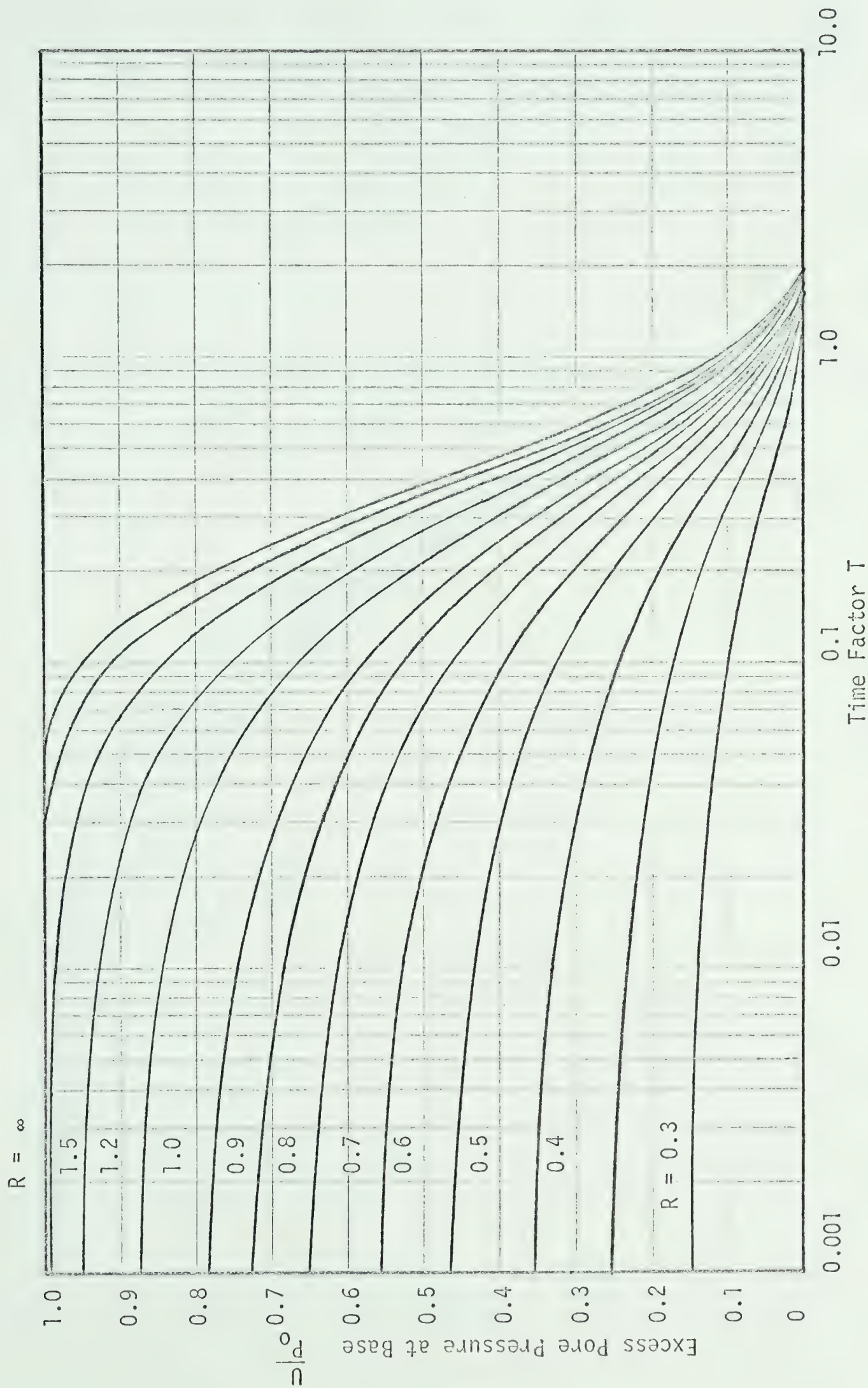


Fig. (2.5) Post Thaw Excess Pore Pressure Dissipation at Base of Sample

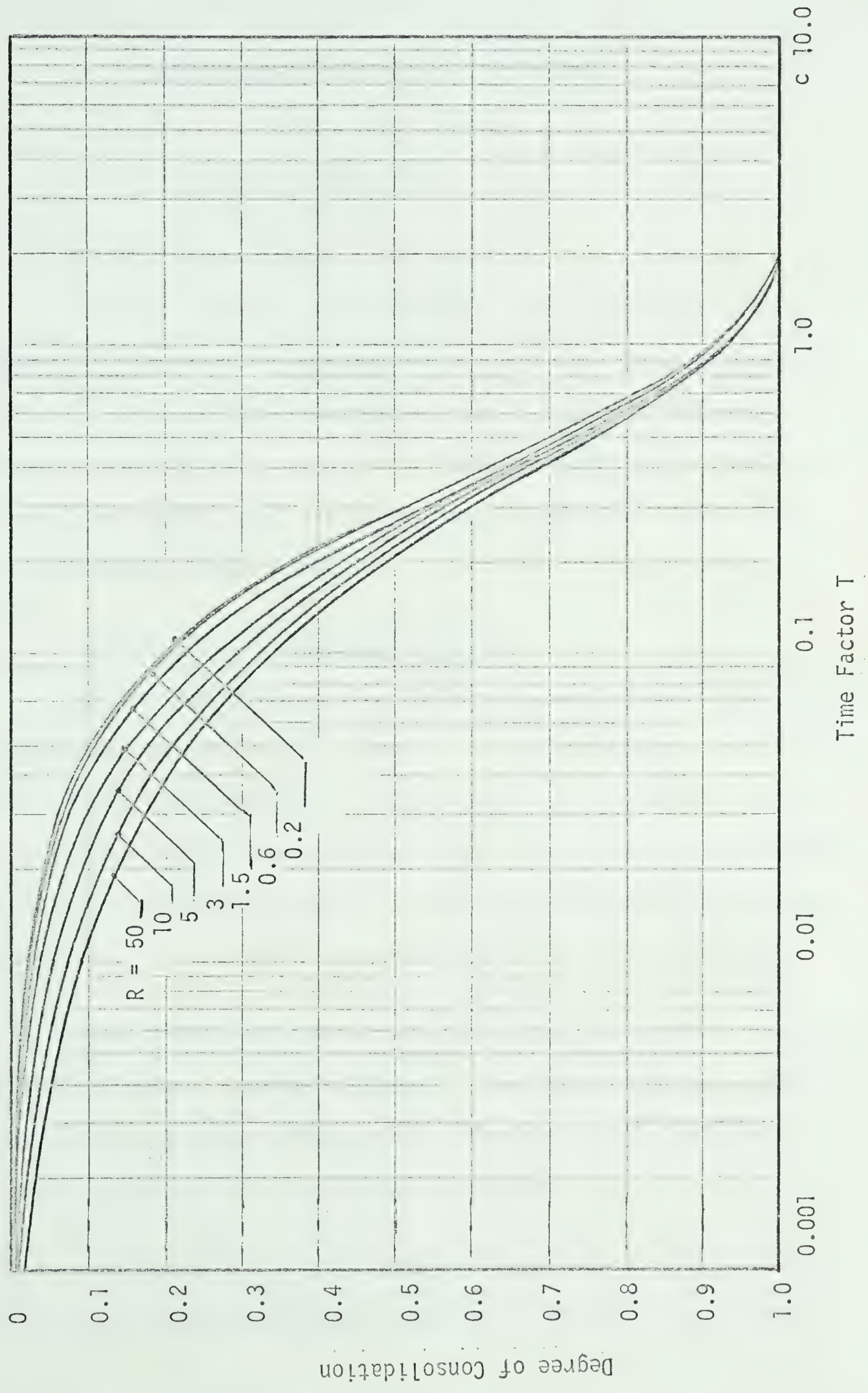


Fig. (2.6) Theoretical Settlement Curves for Post Thaw Consolidation.

CHAPTER III

DESIGN AND DESCRIPTION OF THE PERMODE

3.1 Preliminary Testing

a) Soil Description

The soil used for the preliminary tests was Athabasca Clay. A summary of the mechanical properties of this soil are contained in Appendix A. The soil was selected because of its relatively high compressibility and low coefficient of consolidation ($c_v = 15\text{ft}^2/\text{year}$). Moreover, lean silty clays are more common in Northern Canada than clays with higher liquid limits.

b) Preliminary Consolidation Tests

Preliminary consolidation tests were carried out using standard consolidation apparatus with drainage from top and bottom. A slurry of Athabasca Clay was mixed with distilled water in an electric blender at a liquidity index of approximately 4.0. The slurry was warmed and deaired under a vacuum and stored in air tight containers for use as required.

Three tests were carried out on samples which were initially consolidated under a load of $.02\text{kg}/\text{cm}^2$. Load increments were added up to a maximum pressure of $6.3\text{kg}/\text{cm}^2$. The ratio of $\frac{\Delta P}{P}$ was kept at a value close to 1.0 for all load increments.

A second series of six tests was carried out using a slurry which was prepared as previously described. The slurry was placed

in cylindrical containers 2 inches long and 3 inches in diameter and was frozen in a deep freeze (-14°F). The samples were completely frozen in six to twelve hours. The samples were trimmed to size on a lathe while frozen and placed in the oedometer. The samples were then allowed to thaw at an uncontrolled rate in the oedometer under various initial loads. Once thawing and consolidation was complete under the initial load, subsequent load increments were applied up to a maximum of 6.3kg/cm^2 . In all of the above testing the load increments applied after the initial load were such that the load increment ratio was close to 1.0

c) Test Results

Values for t_{50} for each load increment in the above testing were determined using the log time fitting method. A value for the coefficient of consolidation c_v , coefficient of compressibility m_v and permeability k was computed from the consolidation data for each load increment.

The values obtained for the three tests carried out on soils not subjected to freezing corresponded very closely to each other for each load increment. Typical values from one test are presented in Table 3.1. The corresponding values for a typical test carried out on the samples subjected to one freeze cycle are contained in Table 3.2

The e versus $\log \sigma'$ curve for the tests is given in Fig(3.1). It is apparent from this figure that the application of a freeze thaw cycle to the soil tends to reduce the final void ratio obtained for the same load when the soil is subsequently thawed. That is, the soil

TABLE 3.1

CONSOLIDATION RESULTS
ATHABASCA CLAY NOT SUBJECTED TO FREEZING

Pressure kg/cm ²	2H (inches)	Void Ratio	t ₅₀ (sec)	ΔP (kg/cm ²)	k cm/sec x 10 ⁻⁷	c _v cm ² /sec x 10 ⁻³	m _v cm ² /gm x 10 ⁻³
0.013	0.5998	2.08	1440	0.020	2.2	0.08	3.0
0.033	0.5640	1.90	2040	0.020	1.2	0.05	2.7
0.053	0.5340	1.75	930	0.040	1.8	0.09	2.0
0.093	0.4907	1.52	600	0.087	1.0	0.12	0.89
0.180	0.4529	1.33	570	0.166	0.39	0.11	0.36
0.346	0.4255	1.19	258	0.334	0.45	0.21	0.22
0.680	0.3943	1.03	198	0.666	0.21	0.24	0.09
1.346	0.3712	0.91	114	1.334	0.16	0.36	0.04
2.680	0.3498	0.81	102	2.666	0.08	0.36	0.02
5.346	0.3305	0.70					

TABLE 3.2

CONSOLIDATION RESULTS
ATHABASCA CLAY SUBJECTED TO ONE FREEZE CYCLE

Pressure kg/cm ²	2H (inches)	Void Ratio	t ₅₀ (sec)	ΔP (kg/cm ²)	k cm/sec x 10 ⁻⁷	c _v cm ² /sec x 10 ⁻³	m _v cm ² /gm x 10 ⁻³
0.015	0.5787	1.77	102	0.014	17.0	1.0	1.6
0.029	0.5654	1.71	480	0.030	8.4	0.18	4.6
0.059	0.4879	1.33	132	0.041	1.4	0.57	0.25
0.100	0.4828	1.31	27	0.087	21.0	2.5	0.83
0.187	0.4481	1.14	49	0.196	3.2	1.2	0.25
0.377	0.4264	1.04	120	0.363	0.48	0.46	0.10
0.740	0.4105	0.96	54	0.720	0.71	0.94	0.08
1.460	0.3882	0.86	54	1.460	0.26	0.85	0.03
2.920	0.3711	0.78	60	2.900	0.09	0.70	0.01
5.820	0.3567	0.71					



Fig. (3.1) Void Ratio versus Log Effective Stress - Athabasca Clay

becomes over consolidated as a result of freezing. The difference in void ratios is especially apparent at low pressures. The three frozen samples were thawed under different initial loads. It is apparent from Fig. (3.1) that the magnitude of the load on thawing has no significant effect on the e versus σ' curve obtained.

It has been found that a plot of log permeability versus void ratio will produce a straight line for most soils. (Lambe 1951). Such a plot is presented in Fig. (3.2). Two straight lines were obtained, one for soils subjected to freezing and one for soils not subjected to a freeze cycle. It may be observed from this figure that for a given average void ratio, the permeability can be several orders of magnitude higher at low effective stresses, for a soil which has been subjected to freezing.

This phenomena is evidently due to ice segregation during freezing which causes the formation of primary pore channels on thawing. As the slurry freezes, negative pressures are created at the penetrating frost line (Williams, 1967). Provided the rate of movement of the frost line is not too rapid, the ice will form nuclei and water will be drawn to the ice crystals. In this manner lenses of ice form leaving ribbons or pockets of soil in an over-consolidated state. The segregated ice was readily visible to the eye and was observed in all samples. When the sample is thawed there are, therefore, large primary pore channels left where the ice lenses had formed during freezing. It is these large pore channels which give the soil its very high permeability on thawing. It can be seen from Fig. (3.2) that as higher load increments are applied to the soil which

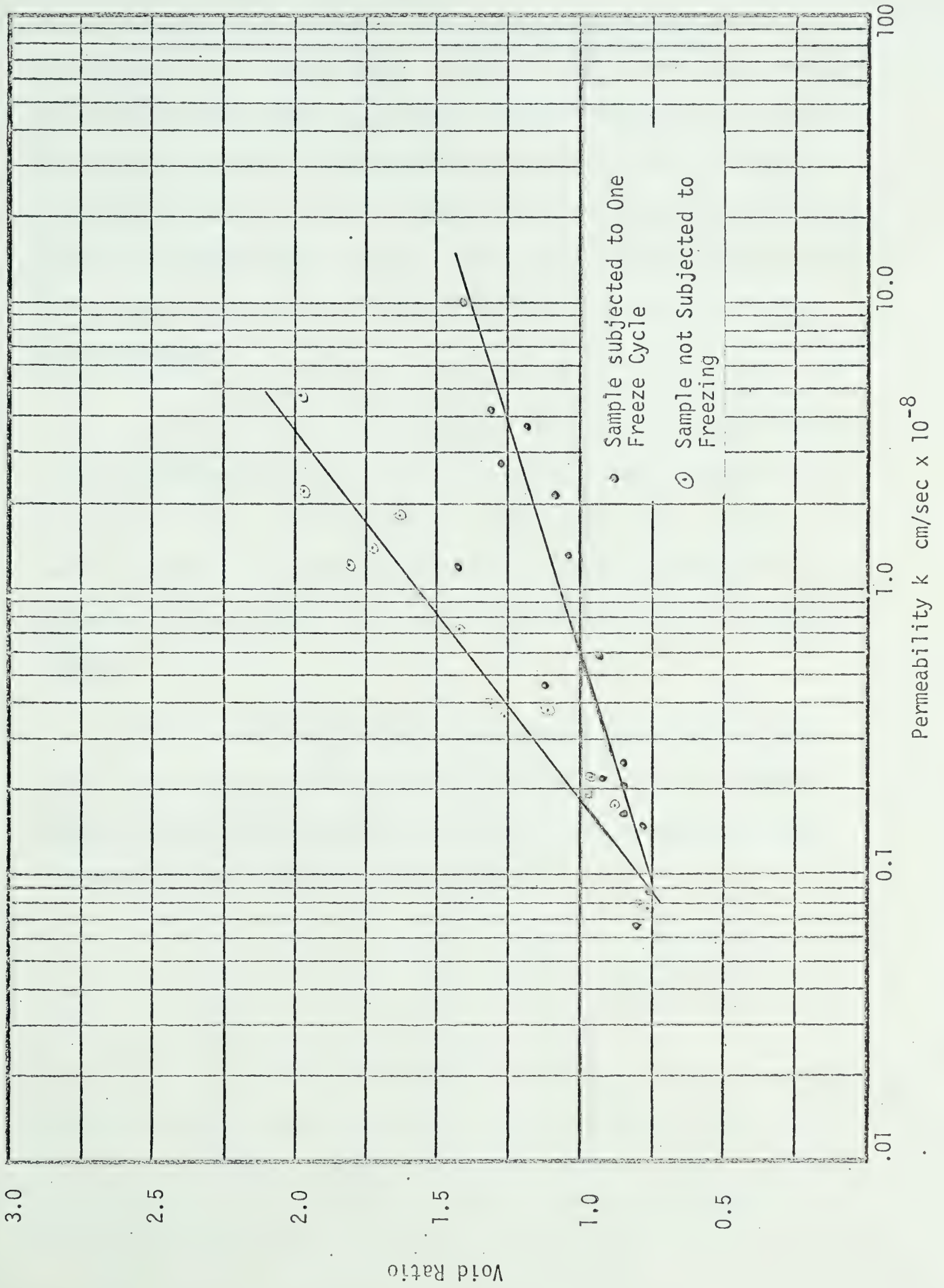


Fig. (3.2) Void Ratio versus Log Permeability - Athabasca Clay

had been subjected to freezing, the primary pore channels are closed up, and the permeability approaches the values obtained for tests on samples not subjected to a freeze cycle. From Fig. (3.2) it can be seen that the permeability of the samples subjected to freezing is essentially the same as that found for samples not subjected to freezing at void ratio of about 0.75. This void ratio corresponds to an effective stress of approximately 1.5kg/cm^2 . This would indicate that the freeze cycle caused the soil to be overconsolidated to a pressure of about 1.5 to 2.0kg/cm^2 .

Williams (1967) has shown that the magnitude of the negative pressures developed at a penetrating frost line depend primarily on the air intrusion value of the soil, which in turn is a function of particle size. It is probable, therefore, that the overconsolidation pressure achieved during freezing is a function of the air intrusion value.

It is felt that further investigation should be carried out in this area to actually measure the magnitude of the negative pressures developed on freezing and relate these pressures to the overconsolidation pressure found on thaw.

3.2 PORE PRESSURE MEASUREMENT DURING CONSOLIDATION

a) Design of the Pore Pressure Measuring System

Before pressures could be accurately monitored in a thaw-consolidation test it was necessary to determine whether or not pore pressures could be accurately measured in a standard oedometer test. Numerous attempts to monitor pressures during an oedometer test have been made by others. The two major difficulties which must be overcome in measuring pore pressures accurately in the oedometer are:

- 1) the effects of compliance of the measuring system on the pore pressures in the soil, and
- 2) the effects of side friction.

A fairly comprehensive theoretical study of the effect of compliance of the measuring system for the oedometer is presented by Perloff et al (1965). Perloff defines the parameter

$$\eta = \frac{AHm_v}{\lambda} \quad (3.1)$$

where A denotes the area of the sample,
 H denotes the height of the sample,
 m_v denotes the compressibility of the soil skeleton,
 λ denotes the compressibility of the measuring system,
 and η denotes the stiffness of the soil skeleton relative to that of the measuring system.

He shows that the measuring system will have a minimal effect on the pore pressures being measured provided the ratio η is high. This is so when the volumetric compressibility of the soil skeleton is high relative

to the volumetric compressibility of the measuring system. It is apparent from equation (3.1) that the effect of system compliance can be reduced by minimizing the volume of fluid in the measuring system and by increasing the volume of soil in the oedometer.

Leonards and Girauld (1961) have investigated the effect of load increment ratio and side friction on pore pressures measured in the oedometer test. Their findings indicate that provided greased teflon is used on the sides of the oedometer and the load increment ratio is greater than a critical value (which varies depending on soil type and previous stress history), the pore pressures can be measured accurately and will conform fairly closely to that predicted by the Terzaghi consolidation theory.

Barden and Berry (1965) investigated the effects of structural viscosity (creep) on the pressures measured in the oedometer tests. Their findings indicate that the effects of structural viscosity can be minimized by using thicker samples and applying load increment ratios greater than 1.0. Provided these conditions are satisfied, the pressures measured in a normally consolidated soil should be in good agreement with the Davis and Raymond (1965) non-linear consolidation theory.

The permode (permafrost oedometer) was therefore designed to meet the requirements for accurate pore pressure measurement. A diagram of the apparatus is shown in Fig. (3.3). The pressure is measured through the use of a strain gauge which is connected to the transducer. Pressures can be measured to an accuracy of .003 psi. Originally a small porous stone was inset at the bottom centre of the permode. It was discovered, however, that better results could be obtained if a

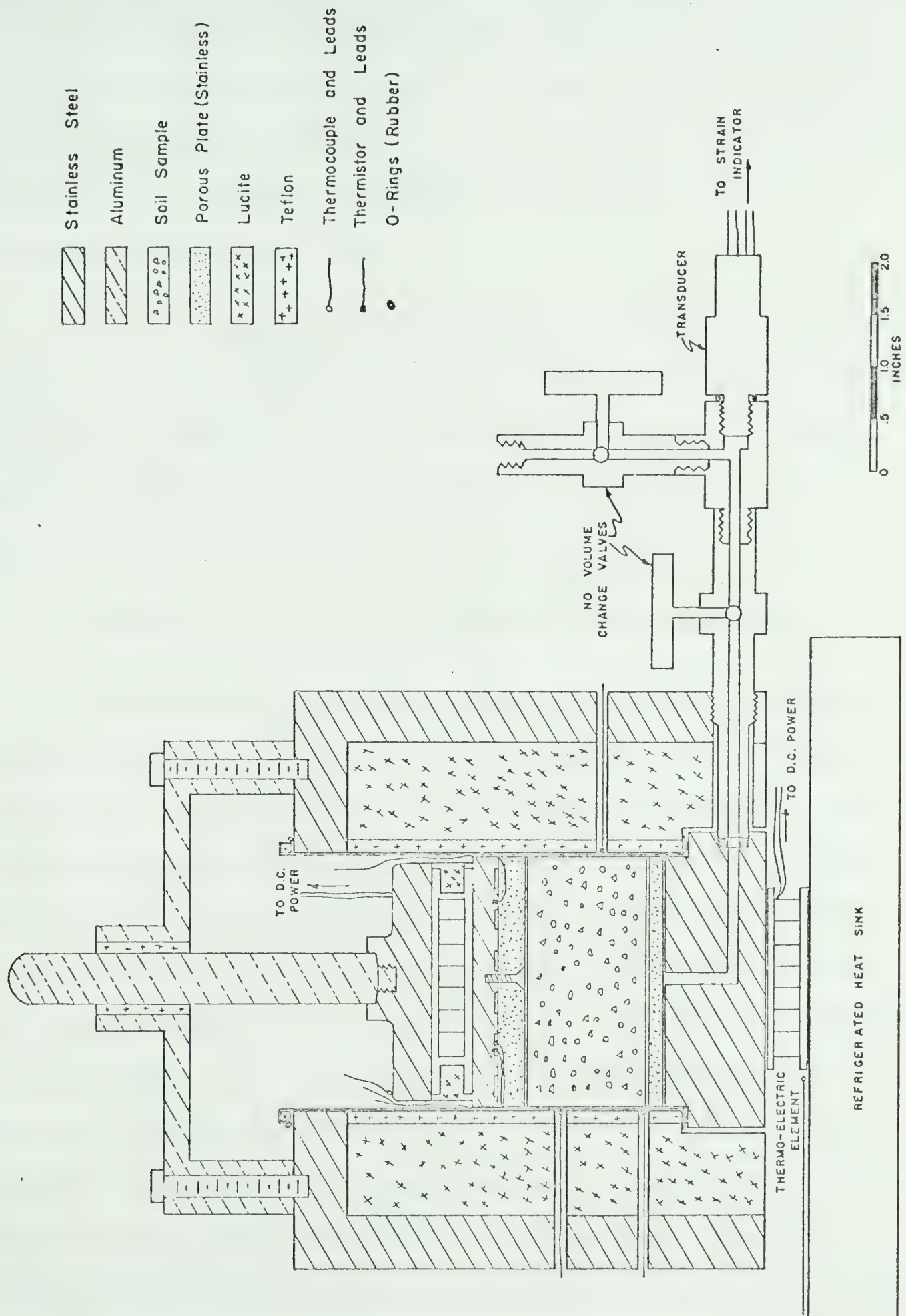


Fig. 3.3 Diagram of the Permode.

circular porous stone which covered the entire base of the soil sample were used. The valves, positioned as shown, permit the re-zeroing of the pressure transducer while a test is in progress, without affecting the pressures at the base of the soil. Re-zeroing was found necessary to compensate for changes in atmospheric pressure which occurred during longer consolidation tests.

The load cap is fitted as loosely as possible without allowing extrusion of the soil around it. As shown in the diagram, a frictionless teflon guide is provided to ensure that the load cap travels smoothly downwards without tilting. This arrangement was found to give much better settlement and pore pressure data during testing.

b) Test Procedure to Confirm Pore Pressure Measurement

The permeometer was assembled with a rubber membrane stretched over the lucite ring which held it firmly in place. The inside and outside of the membrane were thoroughly greased. The measuring system was filled with deaired distilled water. No special procedures beyond the use of deaired water were found necessary to obtain good pressure response. A filter paper was placed over the bottom porous stone and the slurry was deposited in the apparatus under a head of approximately 1/4 inch of water. The load cap was gently lowered into place and the sample was allowed to consolidate under the weight of the load cap ($.02\text{kg/cm}^2$). Once consolidation was complete the rubber membrane was released from the lucite ring. This procedure normally caused an increase in pore pressures which were allowed to dissipate before testing resumed. Various loads were then added to the sample up to a maximum 2.16kg/cm^2 . Time, deflection and pressure readings were taken as for a

standard consolidation test. Various load increment ratios were run on several different specimens. The load increment ratio was varied from 0.8 to 11.4. Pore pressures were also measured after a specimen was unloaded, allowed to obtain equilibrium and then reloaded. The maximum overconsolidation ratio for this type of test was 4.0.

c) Test Results

It was found that the measuring system was sufficiently hard and that very nearly 100 percent pore pressure reaction could be obtained provided a sample of Athabasca Clay of approximately one inch in height was used, (volume 81cc.). It is probable that for soils of lower compressibility a larger volume of soil would be required to obtain good response.

Initially, oedometer tests were carried out without the rubber membrane shown in Fig. (3.3). Typical pore pressure measurements obtained from such tests are shown in Fig. (3.4) and (3.5). The pore pressure curves were normalized to the theoretical Terzaghi curve by determining t_{50} from the settlement data. Thus from

$$c_v = \frac{T H^2}{t} \quad (3.2)$$

$$\frac{c_v}{H^2} = \frac{T_{50}}{t_{50}} = \frac{0.197}{t_{50}} \quad (3.3)$$

and

$$T = \frac{c_v t}{H^2} = \frac{0.197 t}{t_{50}} \quad (3.4)$$

It is apparent from Fig. (3.5) that at the higher loads there is a definite flattening of the pore pressure curves at the outset.

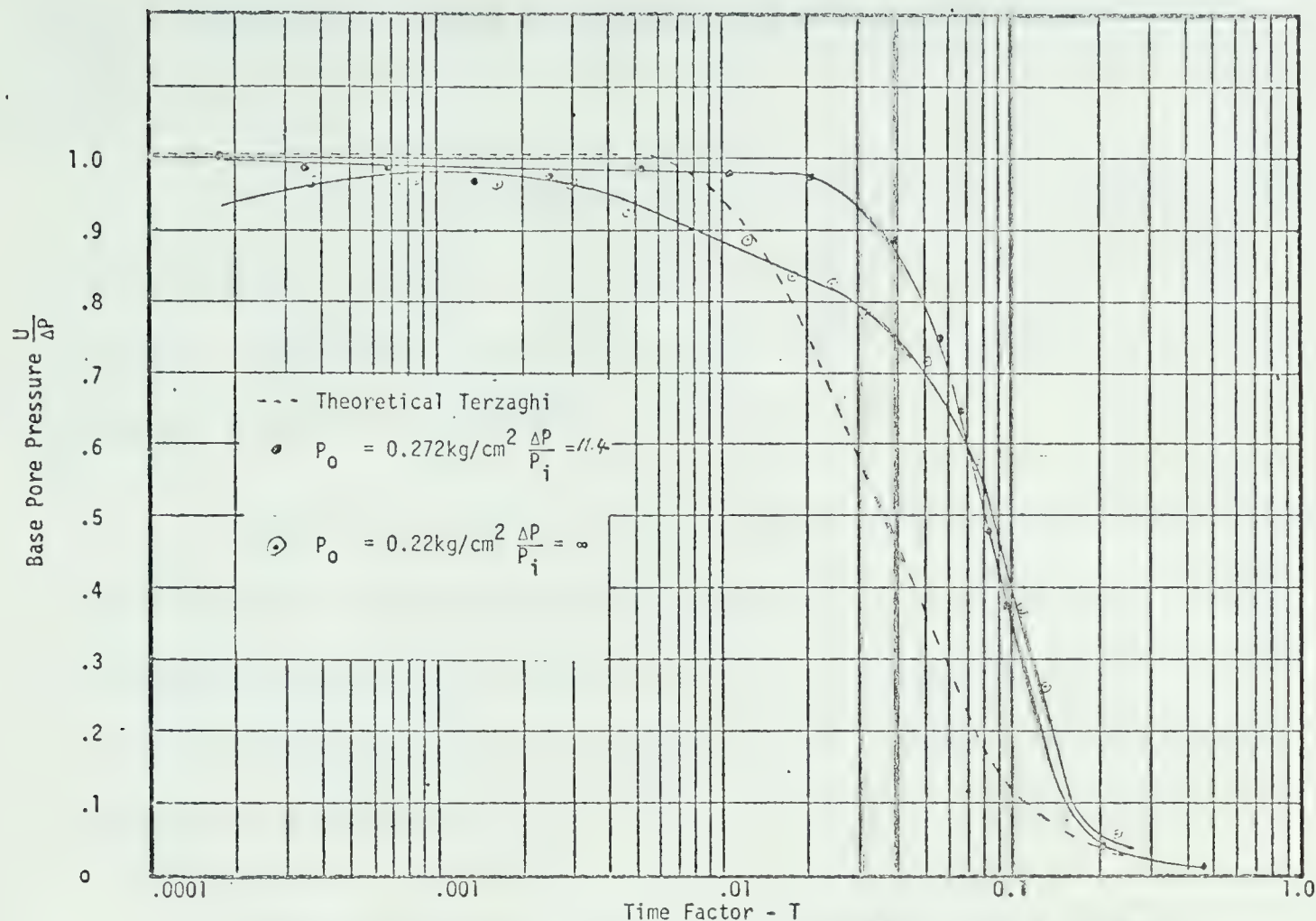


Fig. (3.4) Base Pore Pressure Dissipation Curve for Standard Consolidation - No Membrane

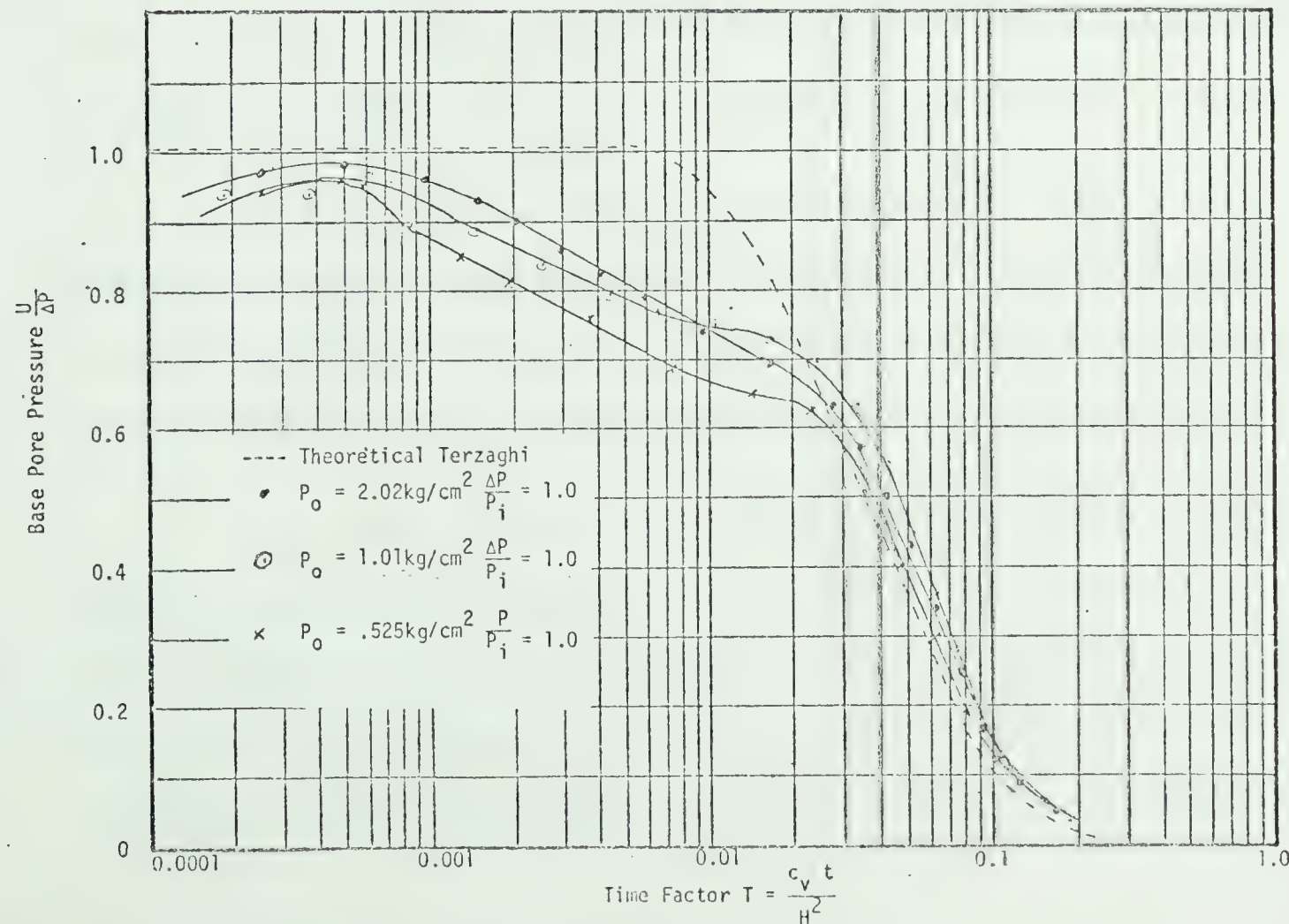


Fig. (3.5) Base Pore Pressure Dissipation Curve For Standard Consolidation - No Membrane

This flattening was found to disappear at the higher load increment ratios ($\frac{\Delta P}{P} > 10$).

It was felt that the flattening may be due to friction and a greased rubber membrane was therefore fitted to the inside of the permeometer. Typical pore pressures measured with the rubber membrane in place are shown in Fig. (3.6), (3.7), and (3.8).

The pore pressures shown in these figures conform very closely with the pore pressures predicted using the David and Raymond (1965) non-linear consolidation theory. It is of interest to note that when the rubber membrane was used without silicone grease, the flattened pore pressure curves as shown in Fig. (3.5) were again obtained.

The pore pressures appear to be unaffected by the load increment ratio for the range of ratios considered, provided the rubber membrane is used. The pore pressure curves also agree with the David and Raymond theoretical pore pressure predictions for several tests performed on overconsolidated soils which had overconsolidation ratios up to 1.8.

It was found that the single most important factor which affected the results obtained was side friction. It is important that the load cap fit as loosely as possible, without allowing extrusion of the soil around it and the rubber membrane must be thoroughly greased.

The results obtained confirm the accuracy of the pore pressure measuring system in the permeometer. It is apparent that further investigation could be carried out to determine the limiting conditions under which pore pressure measurements can be made accurately in the oedometer. Such factors as load increment ratio, overconsolidation

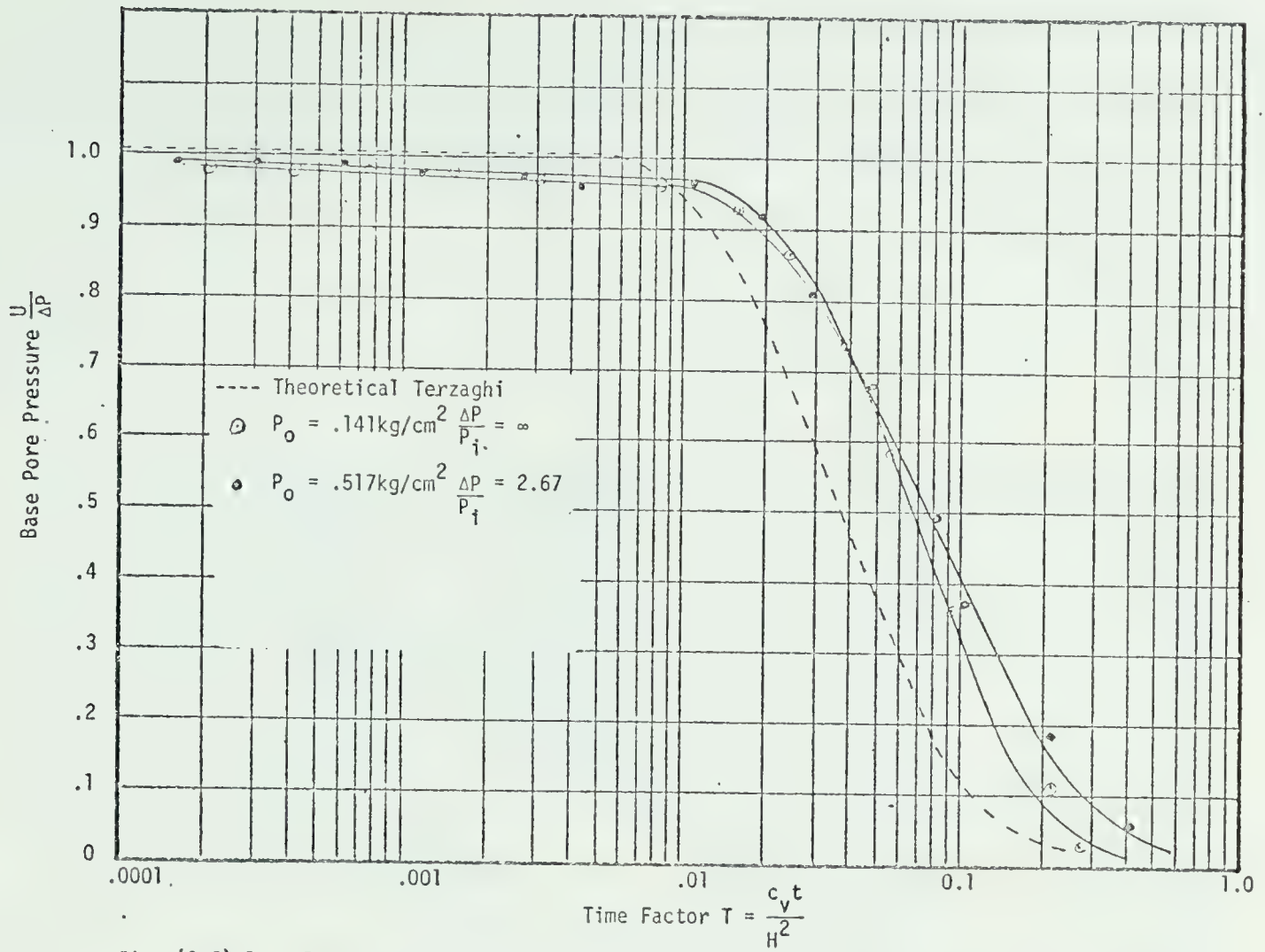


Fig. (3.6) Base Pore Pressure Dissipation Curve for Standard Consolidation - With Membrane

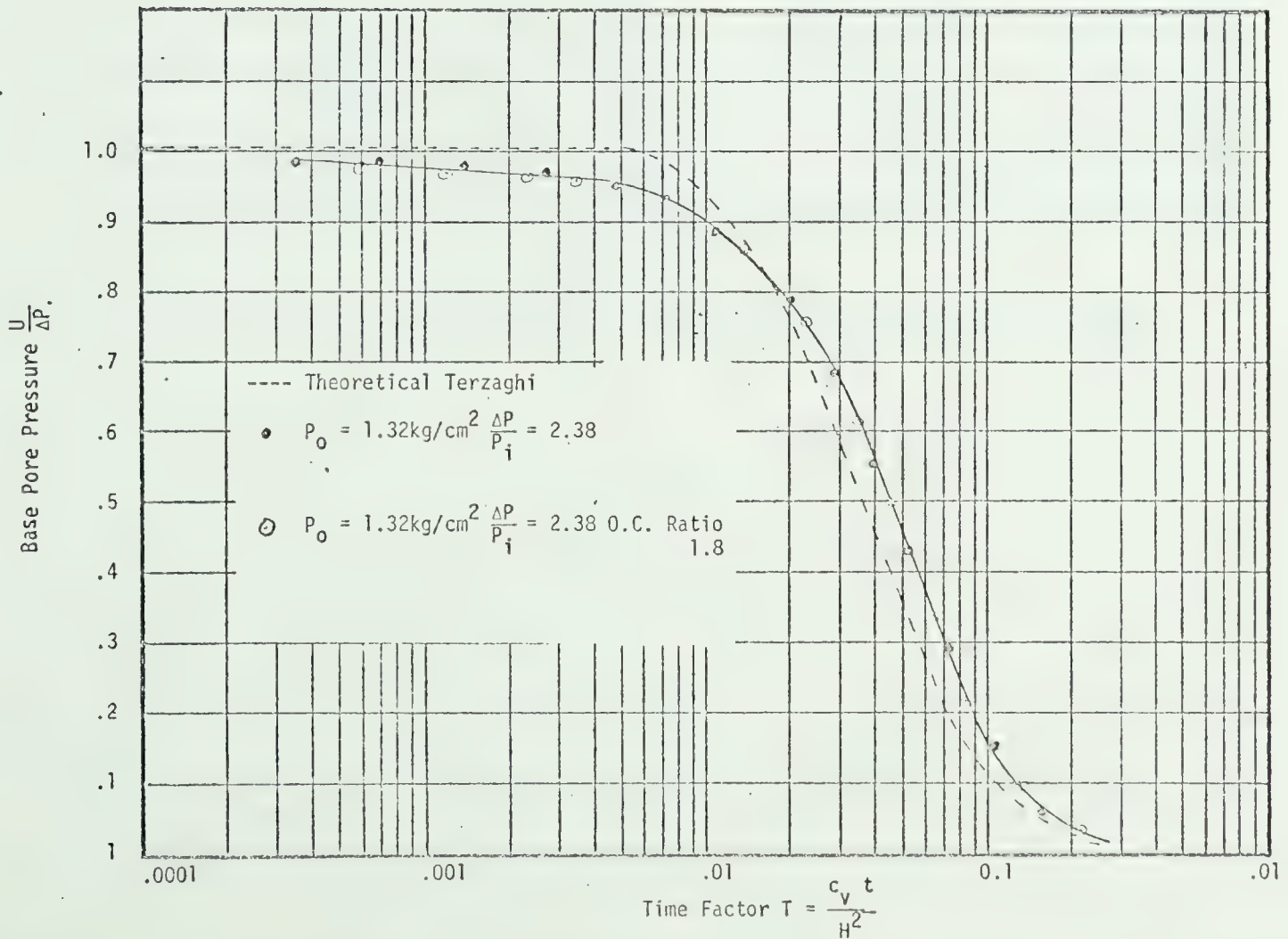


Fig. (3.7) Base Pore Pressure Dissipation Curve for Standard Consolidation with Membrane

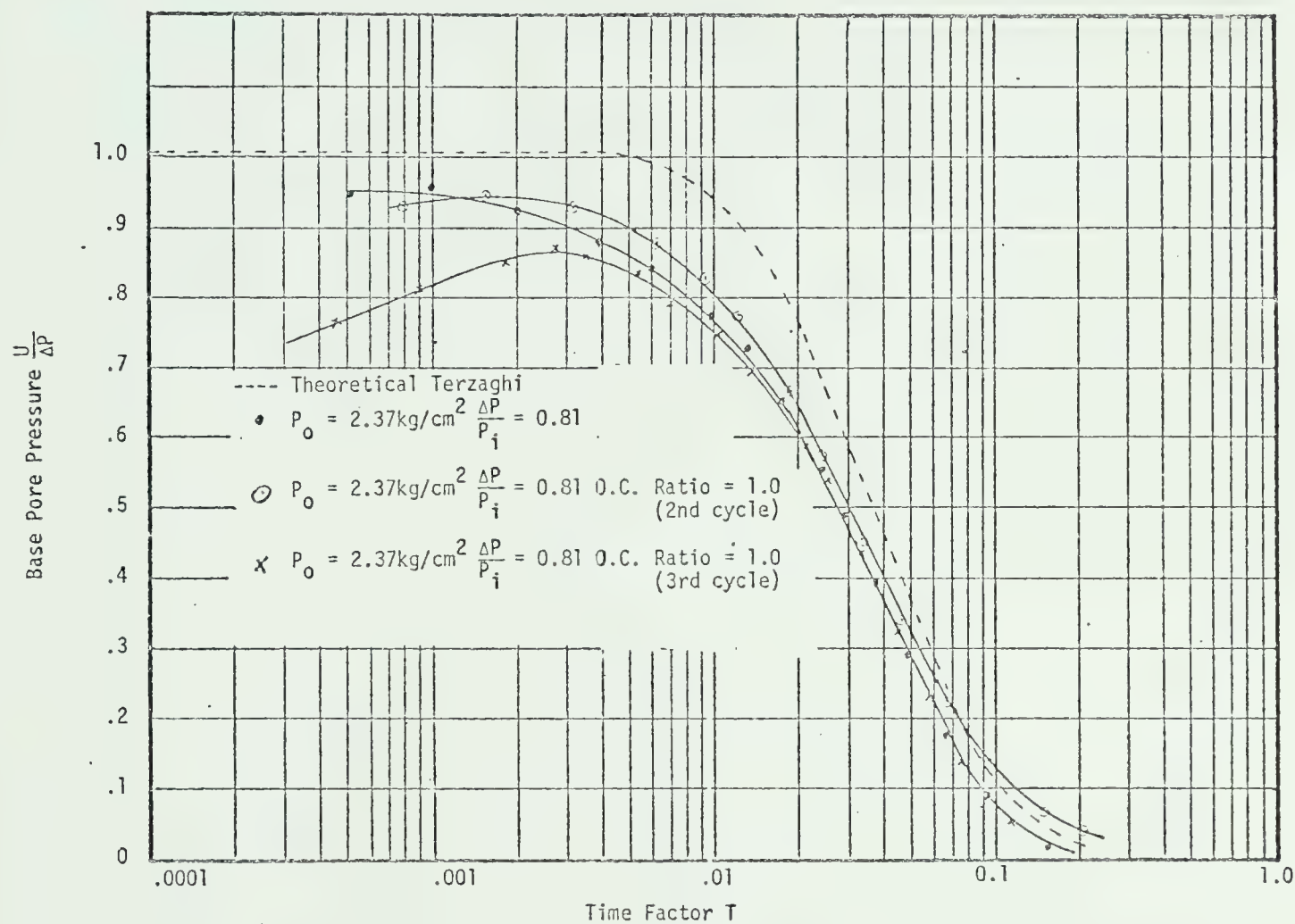


Fig. (3.8) Base Pore Pressure Dissipation Curve for Standard Consolidation - With Membrane

ratio, volume of sample, soil compressibility and so forth will undoubtedly influence the pore pressures measured.

3.3 Description of the Permode

An overall schematic diagram of the thaw-consolidation test layout is shown in Fig. (3.9). The permode itself is shown in Fig. (3.3) and consists basically of a cylindrical lucite ring which is encased in a stainless steel outer ring. The lucite provides insulation and has a thermal conductivity of 5.0×10^{-4} cal/sec°C cc. which is an order of magnitude lower than that for most soils. It was anticipated that sampling of frozen soils could be most conveniently carried out with a sampler with a diameter of 3 inches. The inside diameter of the permode was therefore selected as 2.5 inches since this permits the testing of frozen undisturbed samples carved from a 3 inch diameter core. The length of the inside of the lucite ring is 3.5 inches from the porous stone to the top of the ring. This length permits the testing of high water content samples where large strains occur and also provides a more uniform thermal regime around the sample. The lucite is encased in a steel jacket 0.25 inches thick. This jacket was designed to limit radial deformation to 5.0×10^{-4} inches at an inside pressure of 20kg/cm^2 . It was found that for most thaw consolidation testing, the pressure need not exceed 5kg/cm^2 and therefore, the steel jacket is not really required.

The pore pressure measuring system has been described previously. Freezing of the distilled water in the measuring system during thaw-consolidation tests is prevented by the addition of 50 percent ethelene glycol. The lower porous stone is saturated with ethelene glycol before it is placed in the apparatus.

The temperatures at each end of the sample are controlled

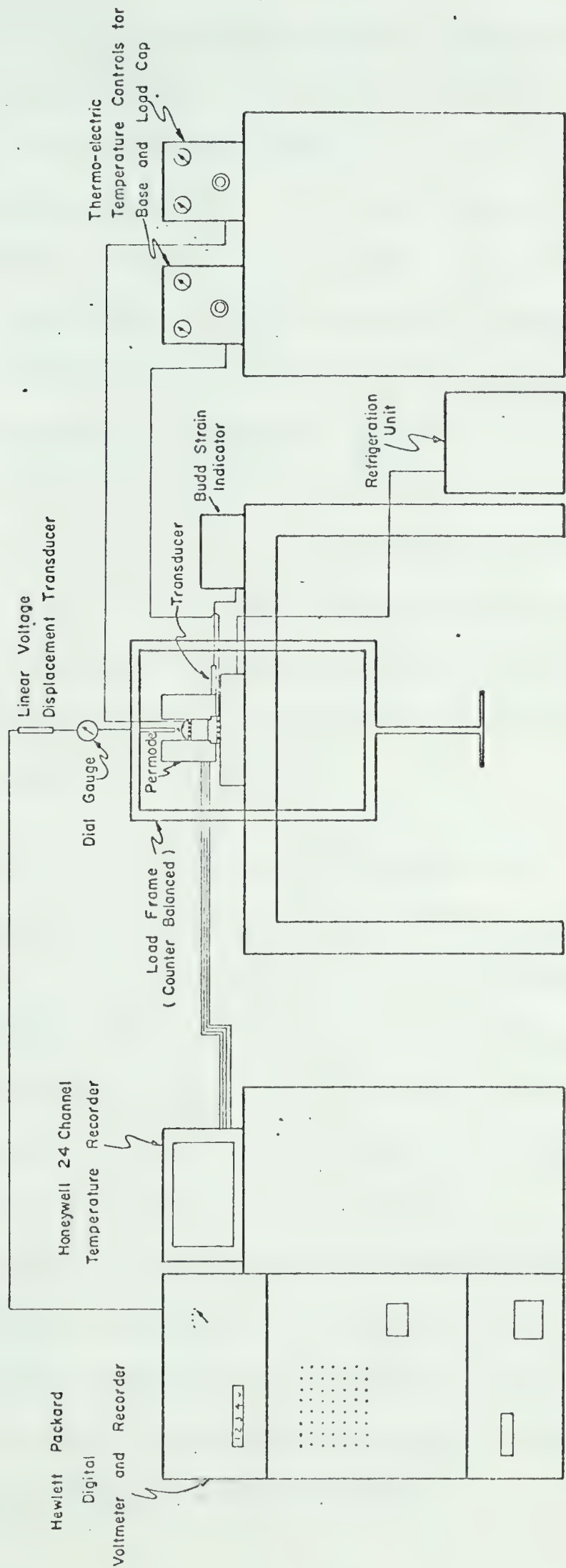


Fig. 3.9 Schematic Diagram of Test Layout for Thaw Consolidation Tests.

through the use of the thermo-electric elements located as shown, one at the base of the permeameter and one in the load cap. The temperature of these elements can be regulated within an accuracy of 0.2°C through the use of temperature controllers. The circuit diagram for a temperature controller is shown in Fig. (3.10). The power output of the controller is regulated by a thermistor sensor through a feedback system. The sensor is placed at the point where temperature is to be controlled and the specific temperature can be established by adjusting the potentiometer to the applicable resistance. It was found helpful to regulate the input voltage to the controller through the use of a Variac, in order to limit the current applied to the thermo-electric elements. The specifications for the thermoelectric elements, temperature sensors and the major components of the temperature controllers are contained in Appendix B.

Four thermo-couples are located as shown, two on the hot sides of the thermoelectric elements and two on the top and bottom of the soil specimen, near the temperature controller sensors. The thermo-couples provide temperature readings to an accuracy of 0.2°C at two minute intervals on a Honeywell Strip Chart temperature recorder. Readings can be taken at various intervals down to a reading every fifteen seconds on this recorder. This system permits a continuous check on the temperatures at these points throughout the test. Additional temperatures are measured to an accuracy of 0.01°C through the use of thermistors located on the sides of the sample at two different heights as shown. These temperatures are recorded at one minute intervals on a digital-voltmeter recorder.

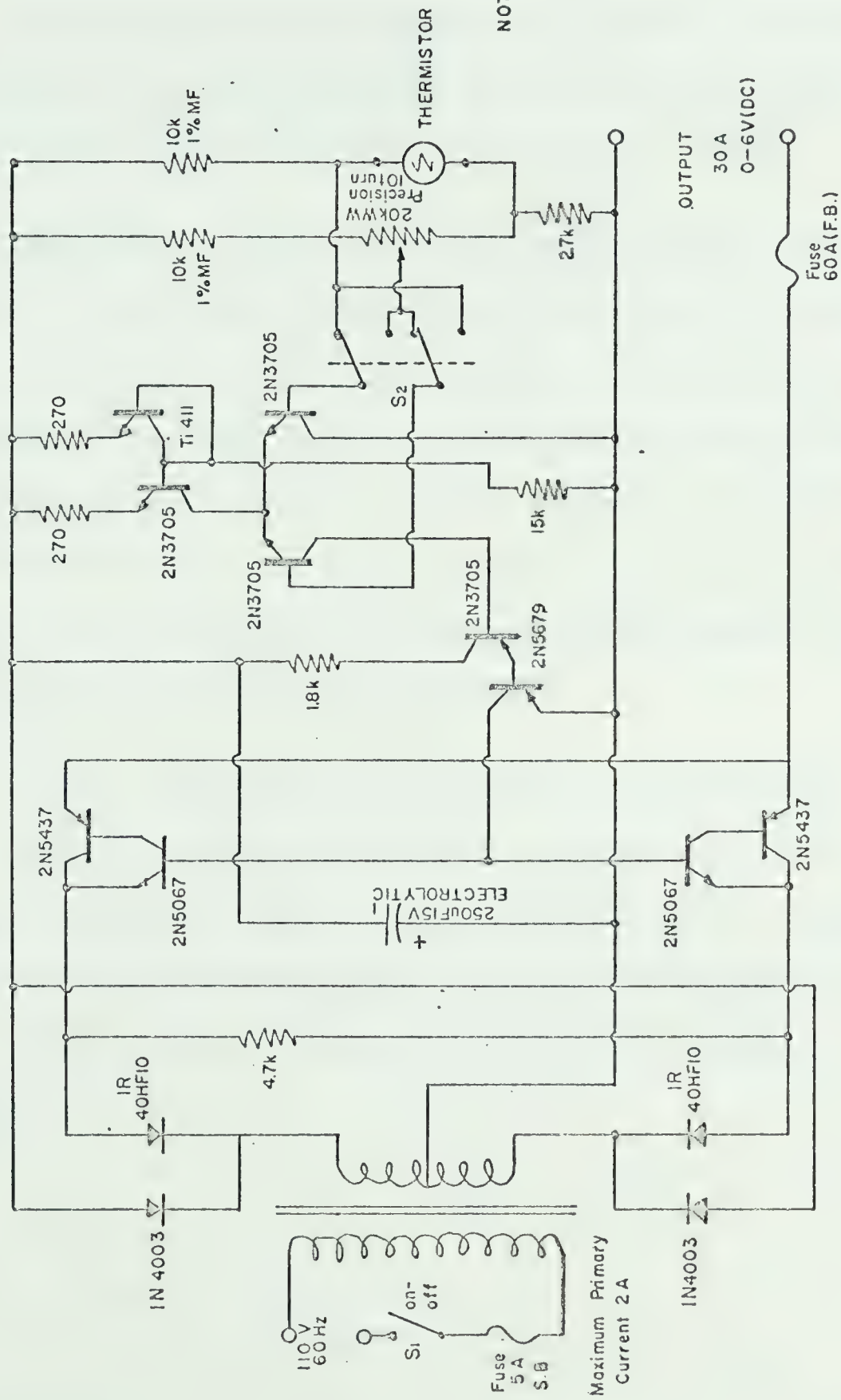


Fig. 3.10 Circuit Diagram for Temperature Controller.

An adequate heat sink is provided for the lower thermoelectric element by placing it directly on a thin flat plate which is cooled by a refrigeration unit. During freezing of the sample a heat sink is provided for the top element by packing the top of the load cap with dry ice. It was found that the presence of air at room temperature around the top of the load cap during thawing of the sample provided a sufficient heat source for the top thermoelectric element.

The sample was loaded directly through a counter-balanced load hanger. It was found that due to the large strains involved in testing frozen soils it was extremely difficult to maintain a constant load on the sample when levered loading systems were used since the lever arm had to be continuously balanced. A constant applied load is necessary in order to obtain accurate pore pressure data. The chief disadvantage of the direct loading system is that the magnitude of the applied loads is limited.

Vertical displacements are read directly with the dial gauge or can be recorded on the digital recorder through the use of an LVDT. Both the dial gauge and LVDT are accurate to .0001 inch. A second dial gauge which is not shown in the figure was positioned so as to provide a direct reading of the sample height to an accuracy of .001 inches.

CHAPTER IV

THAW CONSOLIDATION TESTS

4.1 Test Procedure - Thaw Consolidation

The Athabasca Clay was prepared as a slurry using the procedure described earlier. The permeometer was assembled with the rubber membrane stretched over the lucite ring. The pore pressure measuring system, the bottom porous stone, and the filter paper were saturated with ethylene glycol. The temperature sensors were inserted to contact the membrane and the slurry was deposited in the permeometer under a head of 1/4 inch of distilled water. The specimen was then consolidated under an initial load with the rubber membrane held stretched in place by the ring. Once initial consolidation was complete the rubber membrane was released and the extraneous pore pressures were allowed to dissipate. The specimen was now ready for testing.

The reliability of the pore pressure measurements was normally corroborated by applying a higher load increment and measuring deflection and pore pressure readings as for a standard oedometer test. Once pore pressures were completely dissipated, the transducer was cut off from the bottom porous stone with the valve and freezing of the sample commenced, using both top and bottom elements. Freezing of a one inch sample would normally take approximately two hours. Deflection readings were taken at approximately fifteen minute intervals. The sample was allowed to reach a constant temperature throughout (normally -10°C) and the next chosen load increment was placed on the hanger.

The top element was switched to heat and the step temperature selected (normally $+10^{\circ}\text{C}$ to $+30^{\circ}\text{C}$). The step temperature would normally be achieved within three to six minutes, and would be accurately maintained by the temperature controller throughout the test.

Deflection and temperature readings were taken at two minute intervals. This interval was found convenient and enabled the settlement and temperature curves to be accurately defined. Since the pressure transducer could be damaged by sustained negative pressures, it was necessary to check the pressure at intervals and leave the valve to the transducer turned off until positive pressures were obtained. Once positive pressures occurred, usually just prior to the thaw plane reaching the bottom of the sample, pressure readings could be recorded every two minutes.

The time required to completely thaw a particular sample depends primarily on the sample height, the initial temperature, and the step temperature used in the test. An average length of time for thaw was one to two hours. Readings were continued after thawing was complete and the sample entered the post thawing consolidation phase. Once post thaw consolidation was complete and the excess pore pressures were completely dissipated, a standard consolidation test at a higher load could be carried out, or, the sample could be refrozen and another thaw consolidation test carried out. Once the testing program was completed for the sample, it was removed from the permeometer and the final height, moisture content and weight of dry soil were determined.

4.2 Calculations

The height of the sample at any point in the test is found by adding the difference between the dial reading at that point and the dial reading at the end of the entire test to the height of the sample as measured when the sample was removed from the permeometer at the end of testing. For the tests shown, the final height is the average of twelve random measurements on the sample. The void ratio at any time in the tests will be:

$$e = \frac{H - H_0}{H_0} \quad (4.1)$$

where H denotes the height of the sample at that time,

and H_0 denotes the height of soil solids and is computed from,

$$H_0 = \frac{W_s}{A G_s} \quad (4.2)$$

where W_s denotes the weight of dry soil,

A denotes the area of the oedometer (31.85 cm^2),

and G_s denotes the specific gravity of soil solids (2.65).

The degree of saturation of the sample at the end of testing can be computed from the moisture content determined at the end of testing.

$$S = \frac{\omega G_s}{e} \quad (4.3)$$

where S denotes the degree of saturation,

ω denotes the moisture content of the sample at the end of testing,

and e denotes the void ratio at the end of testing.

The various thaw consolidation parameters were calculated in the same way for each test. As an example, the calculations for test 5 - 4 - 3 are presented. Refer to Fig. (4.1) and Table 4.1. A few words of explanation regarding the pressure, deflection and temperature data presented in chart form in Fig. (4.1) are necessary. The information has been plotted against the square root of time in all cases. The pressure is the positive pressure measured at the base of the sample expressed as a fraction of the total pressure on the sample. The height of the sample at the start of testing prior to freezing is shown and the point at which the post-thaw consolidation phase started is indicated by an arrow.

H_s denotes the height of the sample prior to freezing.

H_f denotes the maximum sample height achieved when frozen.

The void ratios at each stage of the test are given and are denoted as follows.

e_s the void ratio at the start of testing (prior to freezing).

e_f the average void ratio of the sample once freezing

is complete and the sample is at a uniform temperature.

e_e the void ratio at the end of the complete thaw consolidation test. (Post thaw pore pressures completely dissipated).

The temperatures were measured at the same points in all tests.

T_3 denotes the temperature at the top of the sample.

T_{21} denotes the temperature at a point on the side of the sample a distance of 0.6 inches from the bottom

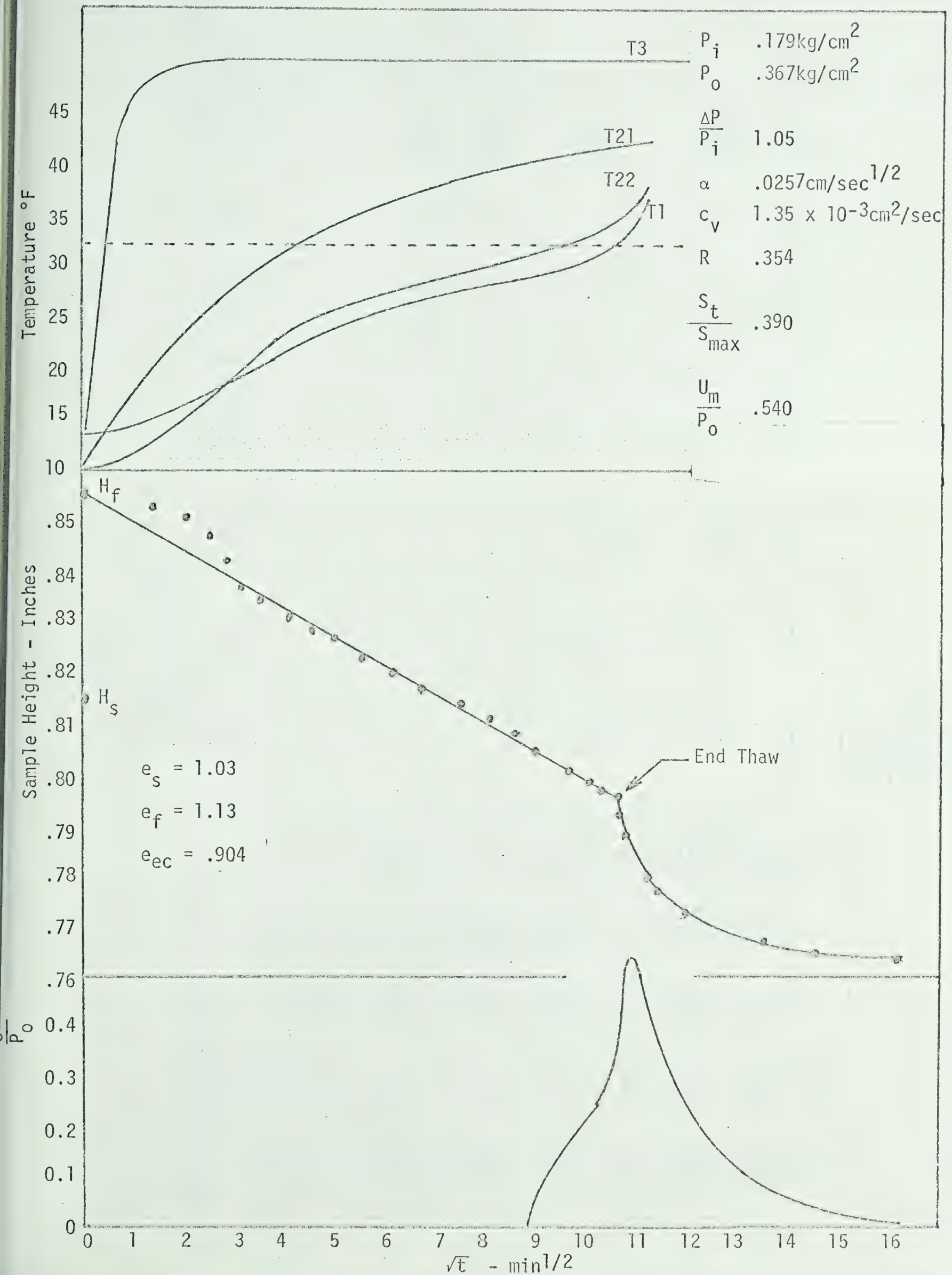


Fig. (4.1) Thaw Consolidation Data Test 5-4-3

TABLE 4.1

SUMMARY OF RESULTS
TEST SERIES 5-4

SOIL - ATHABASCA CLAY

Test	Phase	Press kg/cm ²	Dial in	H in	e	t ₅₀ min	k cm/sec x 10 ⁻⁷	c _{v2} /gm x 10 ⁻³	m _{v2} /gm x 10 ⁻³	$\sqrt{t_{time}}$ (End Thaw) Min ^{1/2}	X (in)	α cm/sec ^{1/2}	R = $\frac{\alpha}{2\sqrt{c_v}}$	$\frac{S_t}{S_{max}}$	$\frac{U_n}{P_0}$
5-4-0	Start	.0245	.7640	1.2737	2.170	-	-	-	-	-	-	-	-	-	-
	End		.4700	.9797	1.447	-	-	-	-	-	-	-	-	-	-
5-4-1	Start	.179	.4700	.9797	1.447	75	2.3	0.25	.92	-	-	-	-	-	-
	End		.3987	.9084	1.265	-	-	-	-	-	-	-	-	-	-
5-4-2	Start	.179	.3987	.9084	1.265	-	-	-	-	-	-	-	-	-	-
	Frozen	.179	.4469	.9084	1.265	-	-	-	-	-	-	-	-	-	-
	End Thaw		.3452	.8549	1.130	22	2.55	0.95	.268	13.9	.9057	.0214	.347	$\frac{.1535}{.0935} = .57$	.61
	End		.3052	.8149	1.030	-	-	-	-	-	-	-	-	-	-
5-4-3	Start	.179	.3052	.8149	1.030	-	-	-	-	-	-	-	-	-	-
	Frozen	.367	.3455	.8552	1.130	-	-	-	-	-	-	-	-	-	-
	End Thaw		.2850	.7947	0.980	13.5	1.45	1.35	.107	10.45	.8249	.0257	.354	$\frac{.0202}{.0514} = .39$	.540
	End		.2538	.7635	0.904	-	-	-	-	-	-	-	-	-	-
5-4-4	Start	.367	.2538	.7635	0.904	-	-	-	-	-	-	-	-	-	-
	Frozen	.367	.2978	.8075	1.010	-	-	-	-	-	-	-	-	-	-
	End Thaw		.2472	.7569	0.890	10	0.74	1.699	.044	10.49	.7822	.0245	.299	$\frac{.0056}{.0177} = .37$	.197
	End		.2361	.7458	0.860	-	-	-	-	-	-	-	-	-	-
5-4-5A	Start	.745	.2361	.7458	0.860	10	0.64	1.637	.050	-	-	-	-	-	-
	End		.2194	.7291	0.794	-	-	-	-	-	-	-	-	-	-
5-4-5	Start	.745	.2194	.7291	0.794	-	-	-	-	-	-	-	-	-	-
	Frozen	1.525	.2514	.7611	0.898	-	-	-	-	-	-	-	-	-	-
	End Thaw		.1924	.7021	0.751	16	0.16	0.902	.018	9.8	.7266	.0243	.407	$\frac{.0273}{.0463} = .59$	.317
	End		.1734	.6831	0.704	-	-	-	-	-	-	-	-	-	-
5-4-6	Start	1.525	.1734	.6831	0.704	-	-	-	-	-	-	-	-	-	-
	Frozen	2.220	.2037	.7134	0.780	-	-	-	-	-	-	-	-	-	-
	End Thaw		.1632	.6729	0.676	21	0.04	0.639	.006	7.9	.6931	.0288	.569	$\frac{.0102}{.0200} = .51$	.152
	End		.1534	.6631	0.654	-	-	-	-	-	-	-	-	-	-

END OF TEST DATA

w = 25.0%
W_s = 86.33gm
Final ht = .6631 in
H_o = .4010 in
S = 101.5%

of the sample.

T_{22} denotes the temperature at a point on the side of the sample a distance of 0.3 inches from the bottom of the sample.

T_1 denotes the temperature at the base of the sample.

The following information is presented in summary form on Fig. (4.1).

P_i the pressure under which the sample was frozen.

P_o the pressure under which thaw-consolidation was carried out.

$\frac{\Delta P}{P}$ the load increment ratio = $\frac{P_o - P_i}{P_i}$

α the thermal constant for the test.

c_v the coefficient of consolidation as computed from the post thaw settlement curve.

R the thaw consolidation ratio for the test.

$\frac{S_t}{S_{max}}$ The ratio of the settlement which occurred during thawing to the total settlement which occurred.

$\frac{U_m}{P_o}$ The maximum pore pressure obtained in the test as a fraction of the total pressure on the sample.

From the settlement curve Fig. (4.1), the time at which the thaw phase reached the bottom of the sample is defined. It will be noted that the peak pressure was measured shortly after this time, and

that the temperature measured at the base of the sample was 32°F at this time. In this case, thawing was completed at $\sqrt{t} = 10.45 \text{ min}^{1/2}$ or $t = 109$ minutes after the step temperature was imposed. This time becomes $t = 0$ for the post thaw consolidation process. The settlement for the post thaw phase can be plotted against the log of time on semi log paper. This has been done in Fig. (4.2). The pore pressure dissipation curve is also shown on this figure. Once post thaw consolidation is completed the time for 50 percent post thaw consolidation is established as for a standard consolidation test. The construction is shown on the figure and value of $t_{50} = 13.5$ minutes has been found.

The coefficient of consolidation can be computed from

$$c_v = \frac{T_{50} H^2}{t_{50}} \quad (4.4)$$

As explained in Chapter II the t_{50} for the pressure distribution which exists at the start of post thaw consolidation has a value of 0.28 as opposed to a value of 0.197 for an initial uniform pressure distribution.

The length of the drainage path is taken as the average height for the post thaw consolidation phase. From Table 4.1 $H = 0.7791 \text{ in.}$

$$c_v = \frac{0.28 (.7791 \times 2.54)^2}{13.5 \times 60}$$

$$c_v = 1.355 \times 10^{-3} \text{ cm}^2/\text{sec}$$

An average value for m_v can also be computed from the post thaw data.

$$m_v = \frac{\Delta e}{1 + e \cdot \Delta \sigma'} \quad (4.5)$$

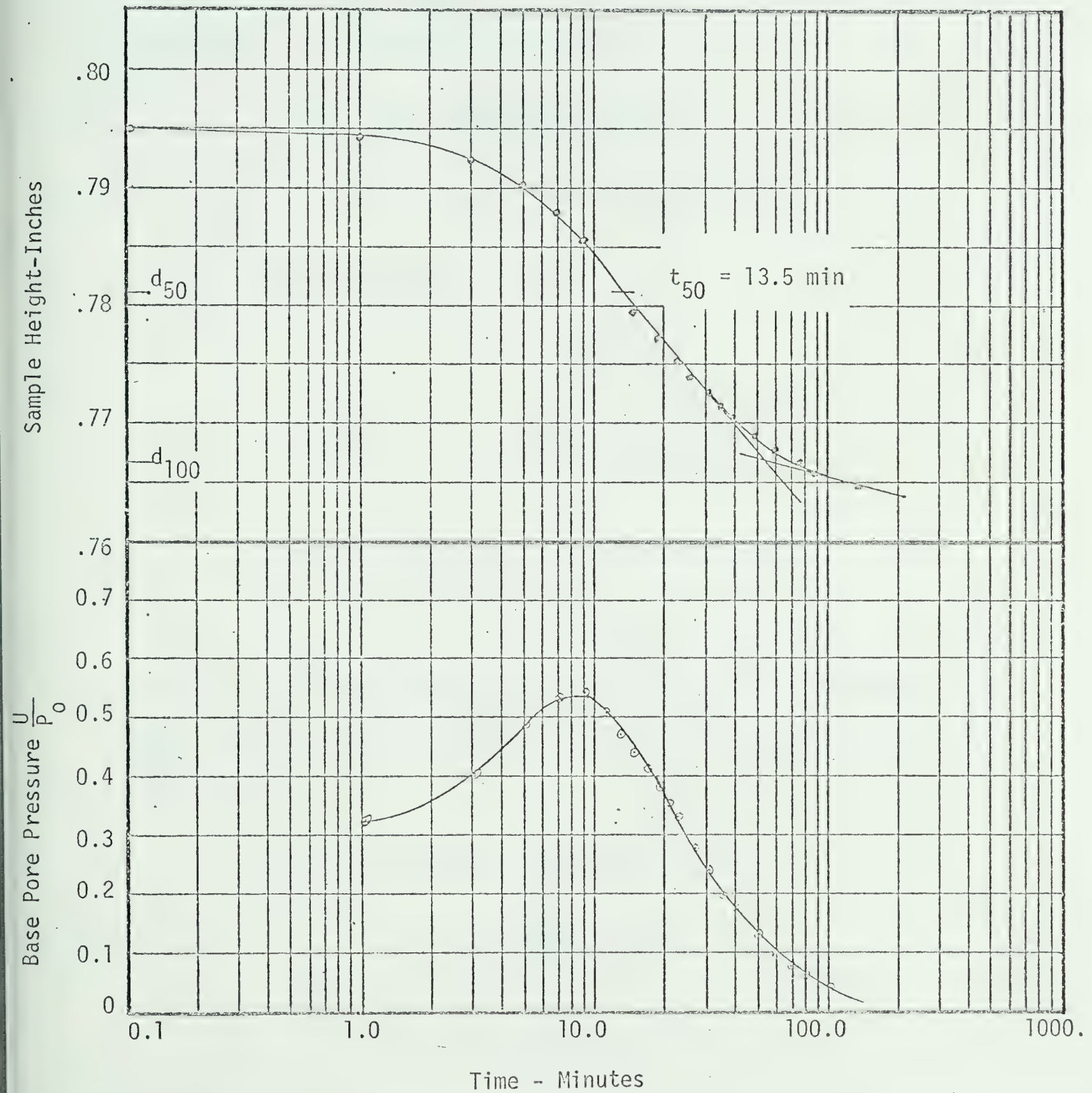


Fig. (4.2) Post Thaw Settlement and Pore Pressure Dissipation Curve
Test 5-4-3

In the case of thaw-consolidation the increase in effective stress ($\Delta\sigma'$) must be taken as the total load applied to the sample. If $\Delta\sigma'$ were taken as the load increment applied to the soil, the compressibility would have a value of infinity where no additional load was added, as for several of the tests.

$$m_v = \frac{.076}{.367 \times 1000 \times (1.94)}$$

$$m_v = .107 \times 10^{-3} \text{ cm}^2/\text{gm}$$

The permeability k is thus

$$k = c_v m_v \gamma_w \quad (4.6)$$

$$= 1.35 (.107) (1) \times 10^{-6}$$

$$k = 1.45 \times 10^{-7} \text{ cm}^2/\text{sec}$$

The thermal constant α is determined from

$$\alpha = \frac{x}{\sqrt{t}} \quad (4.7)$$

where x is the average height of the sample for the thaw phase of the test, and t is the time at which the thaw phase reached the base of the sample.

From Table 4.1, $x = .8249$ inches. Thus

$$\alpha = \frac{0.8249 \times 2.54}{10.45 \sqrt{60}}$$

$$\alpha = .0257 \text{ cm/sec}^{1/2}$$

The value for R can now be computed.

$$R = \frac{\alpha}{2\sqrt{c_v}} \quad (4.8)$$

$$= \frac{.0257}{2\sqrt{1.355 \times 10^{-3}}}$$

$$R = 0.354$$

Since the samples are frozen under a closed system, the degree of consolidation during thawing, $\frac{S_t}{S_{\max}}$ can be computed as follows:

$$\frac{S_t}{S_{\max}} = \frac{H_s - H_{et}}{H_s - H_{ec}} \quad (4.9)$$

where H_s denotes the sample height at the start of testing, prior to freezing,

H_{et} denotes the sample height at the time the thaw plane reaches the bottom of the sample,

and H_{ec} denotes the sample height at 100 percent post thaw consolidation.

Thus, in this example,

$$\frac{S_t}{S_{\max}} = \frac{.8149 - .7947}{.8149 - .7635}$$

$$\frac{S_t}{S_{\max}} = 0.39$$

A value for the maximum pore pressure $\frac{U_m}{P_o}$ is simply

determined from the pore pressure data Fig.(4.1).

In this case,

$$\frac{U_m}{P_o} = .540$$

4.3 TEST RESULTS

a) Test Series 5-4

The soil used for this test series was Athabasca Clay. A summary of the mechanical properties of this soil is presented in Appendix A. A summary of the data obtained from this test series is contained in Appendix C, Table 1. The void ratio versus effective stress for the test series is shown in Appendix C, Fig. (C.1). The details for the stress path followed in this test series are as follows:

Test 5 - 4 - 0

A slurry at a liquidity index of 3.0 was placed in the oedometer and allowed to consolidate under a weight of $.0894\text{kg/cm}^2$. The hanger counterbalance was adjusted to give an initial pressure reading of 100% of the total load on the hanger. Pore pressures dissipated completely within twenty-four hours.

Test 5 - 4 - 1

A load increment of $.0845$ was added to the sample and a standard consolidation test was run to check out the pore pressure measuring system. A response of 98% was obtained and the pore pressures dissipated according to theoretical predictions. There was a significant time lag in achieving the initial 98% response. The settlement and pore pressure curves are presented in Appendix C, Fig. (C.2).

Test 5 - 4 - 2

The sample was frozen under the total load of $.179\text{kg/cm}^2$ and a thaw consolidation test was carried out with no increase in pressure.

Unfortunately the lower thermoelectric element was left on and thawing did not occur proportional to the square root of time. The data for this test is presented in Appendix C, Fig. (C.3).

Test 5 - 4 - 3

The sample was frozen under the total pressure of $.179\text{kg/cm}^2$ and a thaw consolidation test was carried out under a pressure of 0.367kg/cm^2 . The equipment performed satisfactorily. Data for this test is presented in Appendix C, Fig. (C.4).

Test 5 - 4 - 4

The sample was refrozen and a thaw consolidation test carried out with no increase in pressure. The data for this test is presented in Appendix C, Fig. (C.5).

Test 5 - 4 - 5A

A standard consolidation test was run on the sample with the pressure increased to 0.745kg/cm^2 . The data for this test is presented in Appendix C, Fig. (C.6).

Test 5 - 4 - 5

The sample was frozen under a pressure of 0.745kg/cm^2 . A thaw consolidation test was carried out under a pressure of 1.52kg/cm^2 . The data for this test is presented in Appendix C, Fig. (C.7).

Test 5 - 4 - 6

The sample was frozen under a pressure of 1.52kg/cm^2 and a

thaw consolidation test was carried out under a pressure of 2.220kg/cm^2 . The data for this test is presented in Appendix C, Fig. (C.8)

When the sample was removed from the permode it was discovered that the tilt of the load cap was sufficient to allow a height difference of 0.15 inches across the sample. This situation was remedied by installing the teflon guide for the load cap as shown in the drawing of the permode. The time lag in achieving pore pressure response was also discovered to be due to the polyethelene porous disc used at the base of the sample which had a very low permeability. That porous disc was replaced with a more porous stainless steel disc.

b) Test Series 5 - 5

The soil used for this test was Athabasca Clay. A summary of the data obtained from this test series is contained in Appendix D, Table 1. The void ratio versus effective stress diagram for this test series is presented in Appendix D, Fig. (D.1). The details for the stress path followed in this test are as follows:

Test 5 - 5 - 1

A slurry at a liquidity index of 3.0 was placed in the permeometer. The counterbalanced hanger was adjusted to give 100% pore pressure response under an initial load of $.063\text{kg/cm}^2$. The pore pressures dissipated completely within twenty-four hours.

Test 5 - 5 - 2

The applied load was doubled and a standard consolidation test was carried out. The pore pressures rose to 97% of the applied increment and subsequently dissipated very nearly as predicted by the Davis and Raymond theory. The settlement and pore pressure data for this test are presented in Appendix D, Fig. (D.2).

Test 5 - 5 - 3

The slurry was frozen under the total pressure of $.125\text{kg/cm}^2$. A thaw consolidation test was carried out under a pressure of 0.250kg/cm^2 . The data for this test is presented in Appendix D, Fig. (D.3).

During test 5 - 5 - 3 it was discovered that a faulty valve was allowing some anti-freeze to be sucked in at the base of the soil

during freezing. It is unlikely that this affected the results of test 5 - 5 - 3, however, the test series was terminated so that the valve could be replaced.

c) Test Series 5 - 6

The soil used for this test series was Athabasca Clay.

A summary of the data obtained from this test series is contained in Appendix E, Table 2. The void ratio versus effective stress for the test series is shown in Appendix E, Fig. (E.1). The details for the stress path followed in this test are as follows:

Test 5 - 6 - 1

A slurry at a liquidity index of 3.0 was placed in the permeometer. The counterbalanced hanger was adjusted to give 100% pore pressure response under an initial load of $.063\text{kg/cm}^2$. The pore pressures dissipated completely within twenty-four hours.

Test 5 - 6 - 2

The applied load was doubled and a standard consolidation test was carried out. The pore pressures rose to 90% of the applied load increment. The pore pressures dissipated according to theoretical predictions, although there was some flattening of the curve which is believed to be due to friction. The settlement and pore pressure data is presented in Appendix E, Fig. (E.2)

Test 5 - 6 - 3

The sample was frozen under the total pressure of 0.125kg/cm^2 , and a thaw consolidation test was carried out under a load of 0.250kg/cm^2 . The data for this test is presented in Appendix E, Fig. (E.3).

Test 5 - 6 - 4

The sample was refrozen and a thaw consolidation test was carried out under the same pressure (0.250kg/cm^2). The data for this test is presented in Appendix E, Fig. (E.4)

Test 5 - 6 - 5

The sample was refrozen under a pressure of 0.250kg/cm^2 and a thaw consolidation test was carried out at an increased pressure of 0.935kg/cm^2 . The data for this test is presented in Appendix E, Fig. (E.5).

Test 5 - 6 - 6

The sample was refrozen under a pressure of 0.935kg/cm^2 , and a thaw consolidation test was carried out at a pressure of 2.0kg/cm^2 . The data for this test is presented in Appendix E, Fig. (E.6)

Tests 5 - 6 - 7 to 5 - 6 - 11

The sample was refrozen and thaw consolidation tests were carried out under the same pressure (2.0kg/cm^2) for five cycles. The data for each cycle is presented in Appendix E, Figs. (E.7) to (E.11).

There is evidence from the temperature data for this test series that the thermocouple at the base was not functioning correctly. The thermocouple read consistently warmer temperatures than were read on the other temperature sensors attached to the sample. It is thought that this may be the result of the thermocouple not being in proper contact with the soil at the base of the sample.

d) Test Series 5 - 7

The soil used for this test series was Athabasca Clay.

A summary of the data obtained from this test series is contained in Appendix F, Table 2. The void ratio versus effective stress for the test series is shown in Appendix F, Fig (F.1). The details for the stress path followed in this test series are as follows:

Test 5 - 7 - 1

A slurry was placed in the permeometer at a liquidity index of 3.79 ($w = 95.7\%$). The sample was frozen under the weight of the load cap but freezing was carried out before significant consolidation occurred. Thaw consolidation was carried out under a load of $.125 \text{ kg/cm}^2$. A small amount of extrusion occurred during the initial stages of thaw-consolidation. The purpose of this test was to determine whether or not the theory would be valid for soils at very high water contents. The data for this test is contained in Appendix F, Fig. (F.2).

Test 5 - 7 - 2

The sample was frozen under a pressure of 0.125 kg/cm^2 , and a thaw consolidation test was carried out under a pressure of 0.250 kg/cm^2 . The data for this test is presented in Appendix F, Fig. (F.3).

Test 5 - 7 - 3

The sample was frozen under a pressure of 0.250 kg/cm^2 , and a thaw consolidation test was carried out with the pressure increased to 0.50 kg/cm^2 . This test was an attempt to produce a very low R value

by having a very low step temperature increase on the sample (44.5°F). Under these conditions it was found that the time that the thaw plane reached the base of the sample was so poorly defined that the data could not be properly analysed. The data for this test is presented in Appendix F, (Fig. (F.4)).

Tests 5 - 7 - 4 to 5 - 7 - 7

These tests were standard consolidation tests run on the sample. The settlement and pore pressure curves are not included in the data as the results were of little significance. The values for c_v , k and m_v obtained from these tests are given in Appendix F, Table 1.

e) Test Series 5 - 8

The soil used for this test series was Lake Edmonton Clay. A summary of the mechanical properties for this soil is presented in Appendix A. A summary of the data obtained from this test series is contained in Appendix G, Table 1. The void ratio versus effective stress for the test series is shown in Appendix G, Fig.(G.1). The details for the stress path followed in this test series are as follows:

Test 5 - 8 - 1

A slurry was prepared, using the method previously described, at a liquidity index of 2.5 ($\omega = 140\%$). The sample was frozen under the weight of the load cap, but freezing was carried out before significant consolidation took place. Thaw consolidation was carried out under a load of $.063\text{kg/cm}^2$. No extrusion was observed during thaw consolidation. This test was run to determine the validity of the Morgenstern and Nixon theory on a soil at very high water content. The data for this test is presented in Appendix G, (Fig. (G.2)).

Test 5 - 8 - 2

The sample was frozen under a pressure of $.063\text{kg/cm}^2$, and a thaw consolidation test was carried out under a pressure of 0.442kg/cm^2 . The data for this test is presented in Appendix G, Fig. (G.3).

The tests verified the theory for this soil, and it was felt pointless to continue with additional tests. The sample was therefore removed from the oedometer.

f) Test Series 5 - 9

The soil used for this test series was Bentonite which consisted of 80 per cent montmorillonite and 20 per cent illite. All dissolved salts had been washed from the soil and the sample had previously been consolidated to a pressure of $.089\text{kg/cm}^2$ and stored. A summary of the mechanical properties for this soil is presented in Appendix A. A summary of the data obtained from this test series is contained in Appendix H, Table 1. The void ratio versus effective stress for the test series is shown in Appendix H, Fig. (H.1). The details for the stress path followed in this test series are as follows:

Test 5 - 9 - 1

The sample was carved to fit the permeometer and was placed into the apparatus, being careful to eliminate trapped air. The pore pressures were permitted to come to equilibrium under a pressure of $.063\text{kg/cm}^2$. The sample was frozen and a thaw consolidation test was carried out under a total pressure of 0.313kg/cm^2 . The data for this test is presented in Appendix H, Fig. (H.2). The post thaw data is presented in Fig. (H.3).

Test 5 - 9 - 2

The sample was again frozen under a pressure of 0.313kg/cm^2 and a thaw-consolidation test was carried out under a pressure of 0.690kg/cm^2 . The data for this test is presented in Appendix H, Fig. (H.4). The post thaw data is presented in Fig. (H.5).

The tests on Bentonite were carried out primarily to verify the Morgenstern-Nixon theory at high R values. The results obtained verify the theory with the exception of the value for

$\frac{S_t}{S_{\max}}$ obtained for test 5-9-1. The value obtained is considerably

higher than the value predicted by the theory. A moderate amount of extrusion was observed during the thaw phase of this test and is undoubtedly the cause of the high value of $\frac{S_t}{S_{\max}}$ obtained in this particular test.

CHAPTER V

DISCUSSION AND CONCLUSIONS5.1 Validity of the Thaw Consolidation Theory

Examination of the settlement curves for the thaw consolidation tests reveals that settlement is proportional to the square root of time provided the rate of thaw is proportional to the square root of time. This phenomenon was first observed by Tzytovich (1965) and is consistent with the theory developed by Morgenstern and Nixon. That is, the theory predicts that for a constant value of R

$$\frac{S_t}{S_{\max}} = \text{const} = J \quad (5.1)$$

But,
$$S_{\max} = m_v P_o X \quad (5.2)$$

So,
$$S_t = J m_v P_o X \quad (5.3)$$

The volume change associated with the ice-water transformation may be added directly to S_t .

So,
$$S_o = J m_v P_o X + 0.09 n X \quad (5.4)$$

where S_o denotes the total settlement,

and n denotes the porosity of the soil.

From the above equation it is obvious that the settlement must be proportional to the depth to the thaw plane, which moves in this case at a rate proportional to the square root of time.

The Morgenstern and Nixon thaw consolidation theory also predicts that once thawing is stopped, the settlements will proceed at a rate predicted by the classical Terzaghi Theory using as initial values the excess pore water pressure distribution at completion of thawing. Examination of the settlement curves and pore pressure dissipation curves for the post thaw phase of any test reveals that this theoretical prediction is correct.

The maximum pore pressures obtained ($\frac{U}{p_o}$) have been plotted against the $\log R$ in Fig. (5.1). The agreement between the theoretical predictions and the experimental data is remarkable, however, the points are generally higher than the theoretical curve.

It is also possible to plot the base pore pressure dissipation curves for each test. The real time is converted into time factor using the equation

$$T = \frac{0.28t}{t_{50}} \quad (5.5)$$

where t_{50} is determined from the post thaw settlement curve. The values for $\frac{U}{p_o}$ are then plotted against $\log T$ as has been done in Fig. (5.2), for a sampling of R values selected from the test data. The majority of the dissipation curves obtained in testing were found to behave in much the same way as the curves shown in Fig. (5.2). It will be observed from this figure that although the shape of the experimental curves approximates the shape of the theoretical curves, the experimental curves are shifted to the right. This same phenomenon is observed in pore pressure curves obtained from standard consolidation tests, where the actual pore pressure curves are shifted to the right of the theoretical Terzaghi curve. It is possible that the shift of

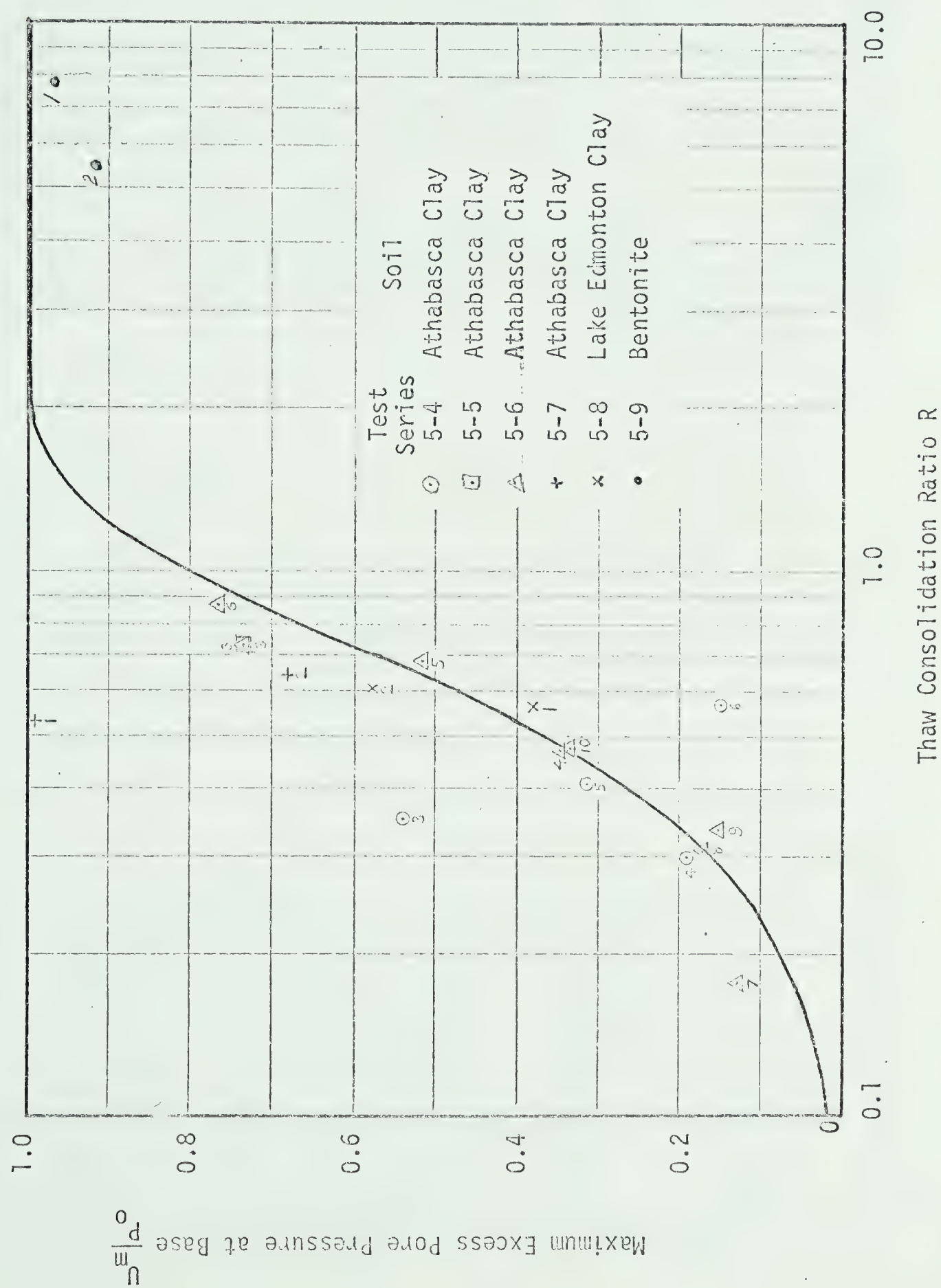


Fig. (5.1) Maximum Base Excess Pore Pressure versus Log R

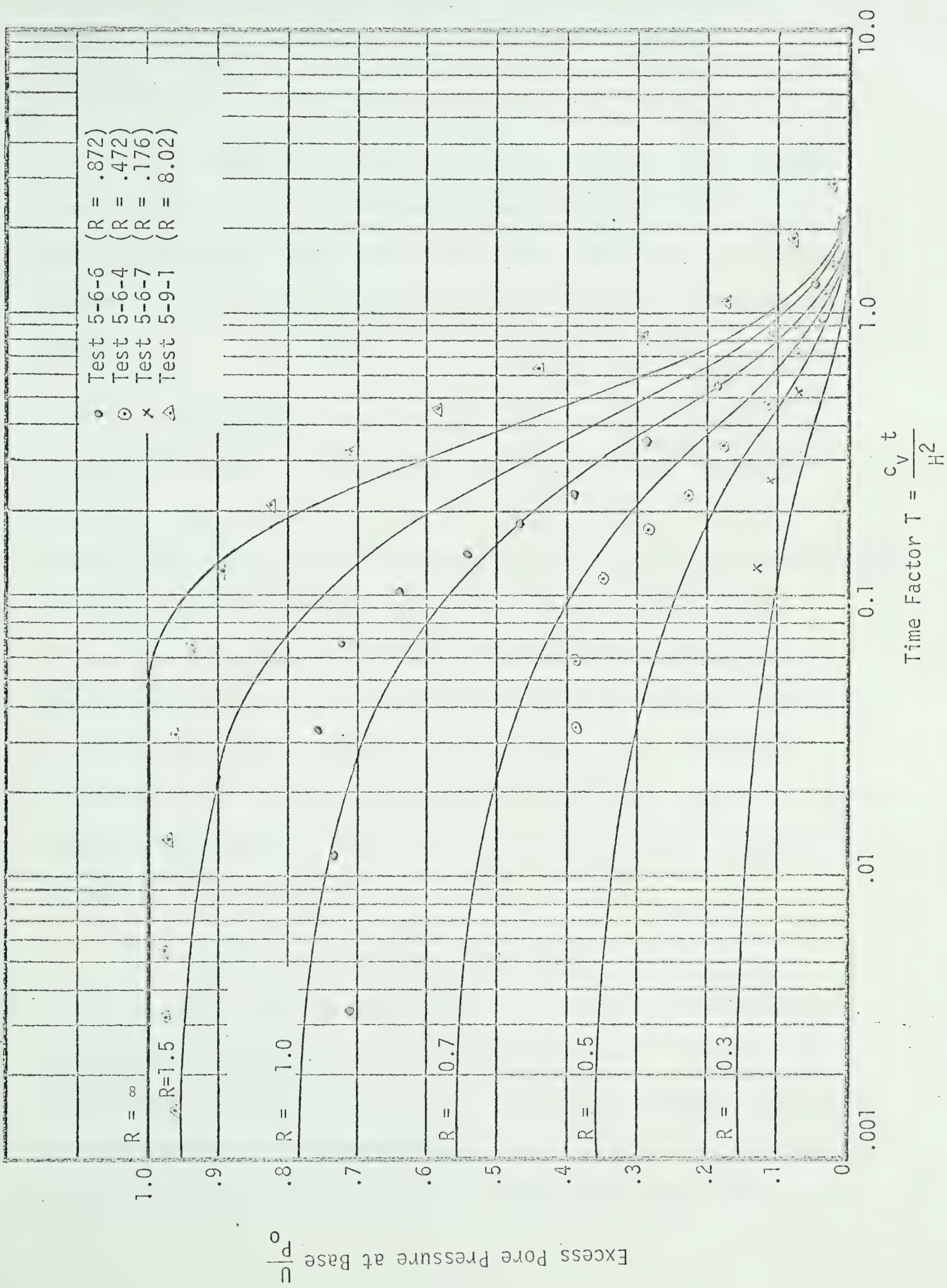


Fig. (5.2) Post Thaw Excess Pore Pressure Dissipation at Base

the dissipation curves found in post thaw consolidation may be due to nonlinearity of the void ratio versus effective stress relationships.

The degree of consolidation $\frac{S_t}{S_{\max}}$ has been plotted against $\log R$ in Fig. (5.3). The agreement between the experimental data and the theoretical prediction is only fair in this case. Possible reasons for the scatter of points on this figure will be discussed later.

Examination of the e versus $\log \sigma'$ curves for test series 5-4 and 5-6 reveal that the void ratio is significantly reduced as successive freeze-thaw cycles are applied even though the load on the sample remains constant. This is probably due to local overconsolidation which takes place during the freezing process when high negative pressures are generated in the soil. Presumably the sample swells on thaw but does not return to its original void ratio overall. There is, therefore, excess water available, which flows out of the sample according to the standard consolidation process. The phenomenon explains why the pore pressures on thawing can be higher than the load increment imposed in the test.

It is apparent, therefore, that the value of σ_0' , the effective stress prior to consolidation, must be a function of the pressure to which the soil is overconsolidated during freezing and the coefficient of swelling for the soil. In the pore pressure calculation for these tests it was assumed that σ_0' was zero in all cases and hence all pore pressures were expressed as a percentage of the total load on

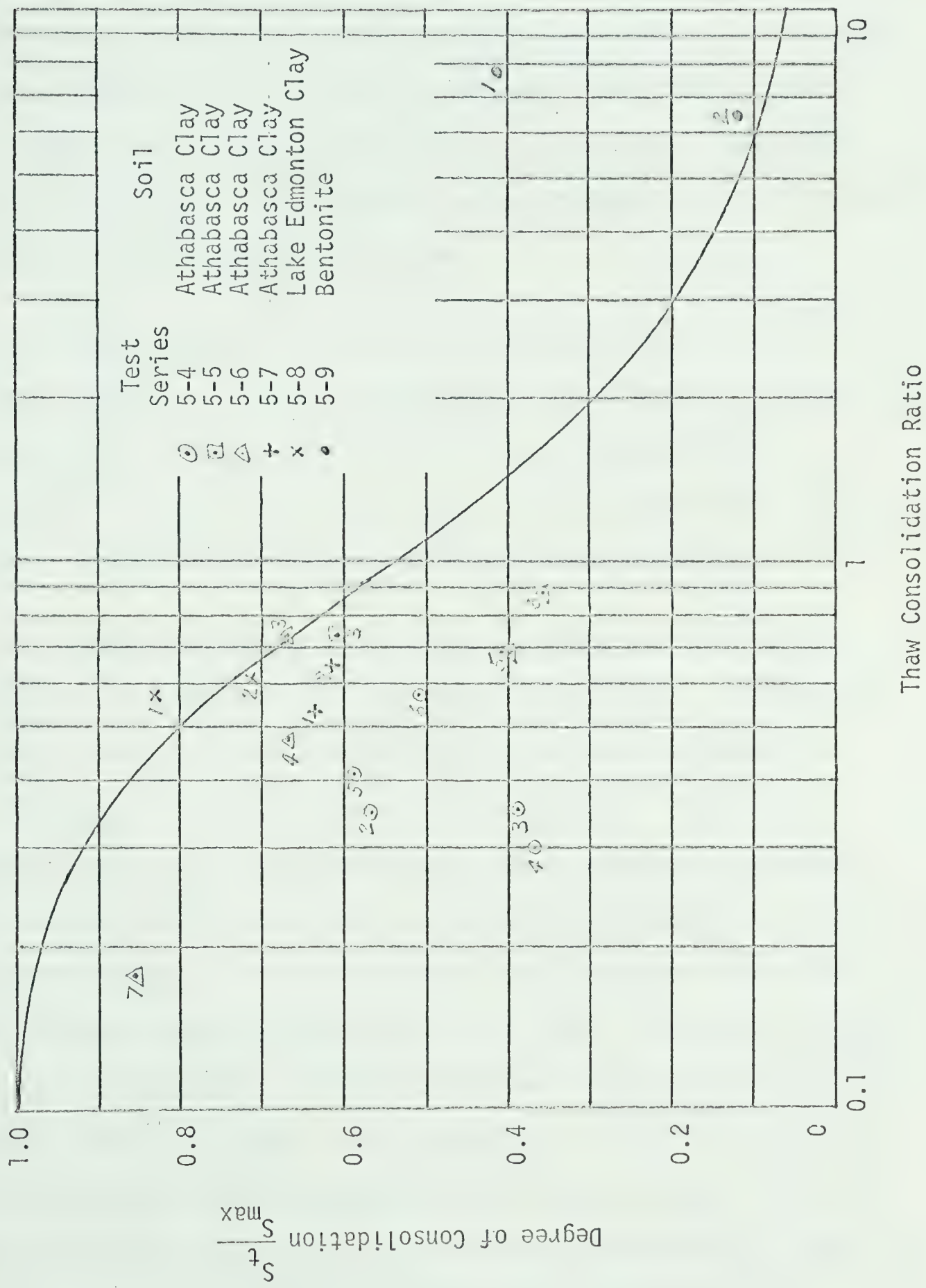


Fig. (5.3) Degree of Consolidation versus Log R

the sample. The data shows that this assumption was essentially correct for the soils tested. It is conceivable, however, that for soils subjected to freeze-thaw cycles where the negative pressures generated during freezing are insufficient to cause a high degree of overconsolidation, or where the coefficient of swelling is very nearly equal to the coefficient of consolidation, then σ_o' could have a magnitude almost as high as the final effective stress and hence the pore pressures achieved on thawing would be much lower.

The prediction of pore pressures during thaw consolidation requires, therefore, a better understanding of the negative pressures generated during freezing and the role these pressures play in determining the initial effective stress on thaw. Considerable work has been carried out by Williams (1967) and others in this area.

Special attention should be paid to the data obtained from tests 5 - 7 - 1 Appendix F, Fig. (F.2), and test 5 - 8 - 1 Appendix G, Fig. (G.1), which were thaw consolidation tests carried out on soils which were initially at very high void ratios. It will be noted that despite the large strains involved, (approximately 50% strain), the resulting settlement curves display the general form predicted by theory. The fact that this should occur is significant in that the theory assumes overall strains are small. The values for $\frac{S_t}{S_{max}}$ obtained from these tests agrees reasonably well with theoretically predicted values Fig. (5.3). From Appendix F, Fig. (F.2) it will be seen that the pore pressures behaved quite erratically just after the maximum pore pressure was read. The phenomena was observed to occur to a very minor degree in several of the other tests run at lower void ratios. It is thought to be due to the collapse of the soil

structure in a localized pocket where a large ice crystal had formed. Such a collapse would naturally be accompanied by a rapid settlement and a significant rise in pore pressure.

The maximum pore pressure achieved in test 5 - 7 - 1 was 99.2% of the total load. This pressure is much higher than the theoretically predicted value of 29% based on the R value of 0.431. There are several possible explanations for this discrepancy.

First of all, the value for c_v which was used to calculate R was determined from an erratically behaved post-thaw settlement curve Appendix F, Fig. (F.2). This results in a value for c_v which is not completely reliable. The values for c_v have been plotted against void ratio for all the thaw consolidation tests performed on Athabasca clay (See Fig. 5.4). The variation of c_v with void ratio has also been plotted for two other soils, Chicago clay (Taylor, 1948) and Tyee River clay (Chattopadhyay, 1969) and these curves provide a better indication of the behaviour one can expect in c_v at different void ratios. The points obtained from thaw consolidation tests on Athabasca clay show considerable scatter, however, a fairly definite trend results as indicated by the curve. The trend indicates that c_v should remain very nearly constant as void ratio increases, however, further tests should be carried out at high void ratios to confirm this. The value for c_v obtained from test 5 - 7 - 1 (circled) is clearly anomalous and should have a value of approximately $1.0 \times 10^{-3} \text{ cm}^2/\text{sec}$. If this latter value is used to calculate R then a value for R of .935 is obtained. This value for the thaw consolidation ratio predicts a maximum pore pressure at the thaw plane of 75% of the total load

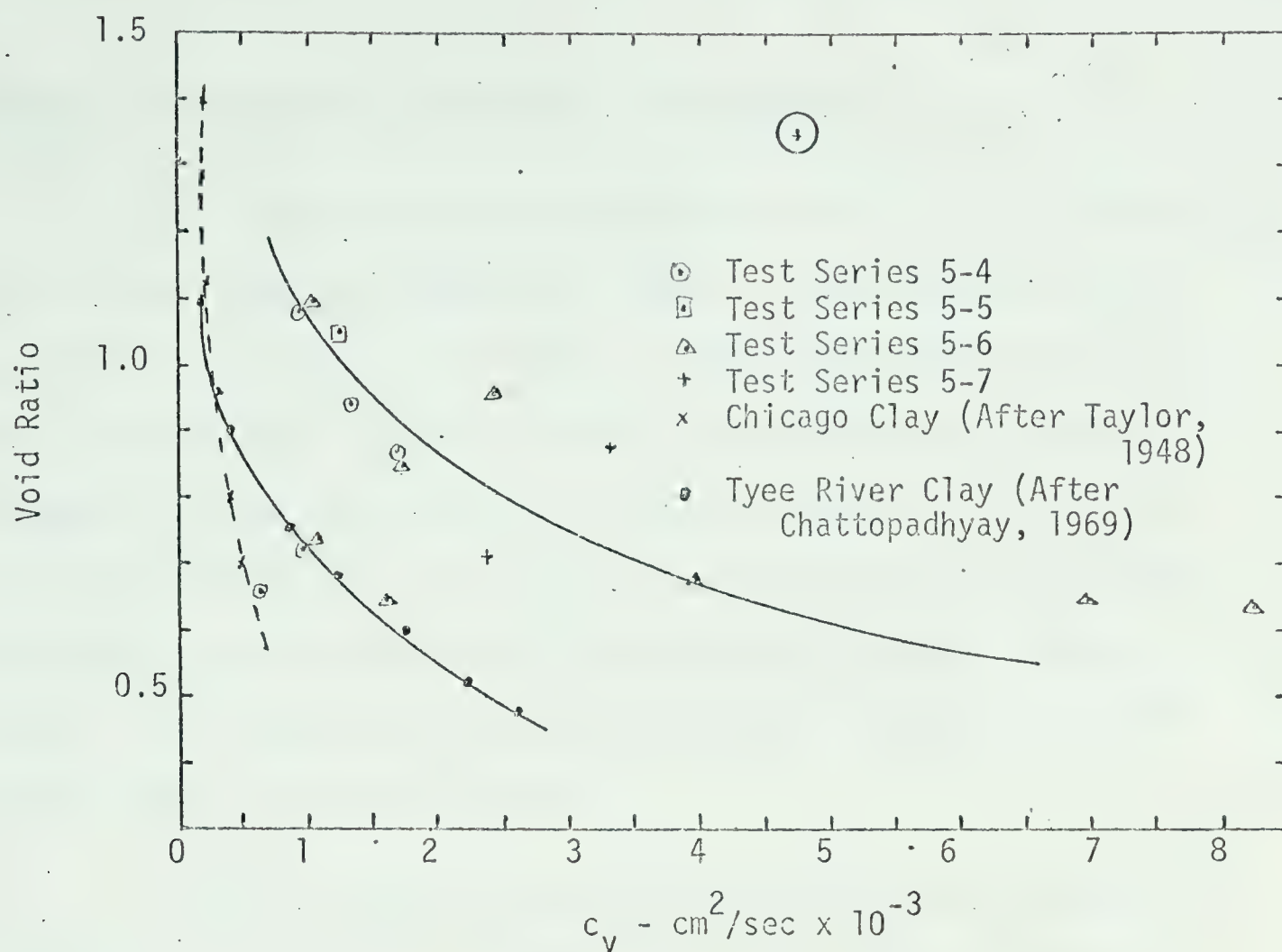


Fig. (5.4) Void Ratio versus Coefficient of Consolidation

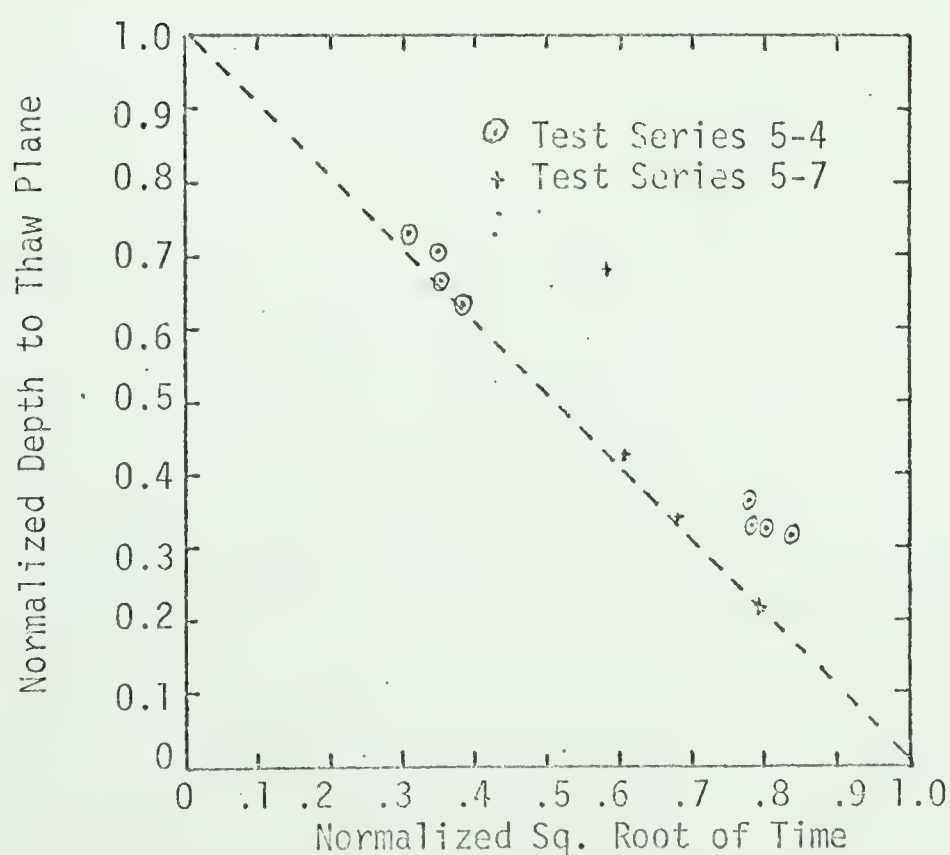


Fig. (5.5) Normalized Depth of Thaw Plane versus Normalized Square Root of Time

which is closer to the experimental value obtained.

The high pore pressures obtained for test 5 - 7 - 1 might also be due to the non-linearity of the $e - \sigma'$ relationship which is known to be increasingly non-linear at lower effective stresses. For very high ice content soils, in fact, it is probable that considerable change in void ratio would have to take place before there was any increase in effective stress. Such a condition would act to maintain pore pressures for longer periods than predicted using a linear $e - \sigma'$ model. It is important to note that the present theory is in error on the unsafe side in this respect.

It is strongly recommended that several thaw consolidation tests be run on soils at very high void ratios before any definite conclusions are drawn regarding the validity of the Morgenstern-Nixon thaw consolidation theory for soils under these conditions.

5.2 Sources of Error

The relative deflection can be measured sufficiently accurately with the dial gauge where overall settlements ($S_{\max}$) are at least 0.01 inches. For tests where the overall settlement is less than this value, the dial gauge positioned as it is to read deflections of the load hanger, did not give reliable readings. It is felt that the scatter of results on the $\frac{S_t}{S_{\max}}$ versus log R curve is due partly to the unreliability of the dial gauge readings. This is especially true where the thermal component of settlement becomes large with respect to consolidation settlement. For example, in test 5 - 6 - 11, S_t was found to be 0.0007 inches and $S_{\max}$ 0.0045 inches. A very small vibration or movement will change the ratio $\frac{S_t}{S_{\max}}$ by a substantial factor.

It was also discovered that temperature changes above the freezing point in the soil will cause the height of the sample to change. As the temperature dropped from room temperature (75°F) to near freezing, the height of the sample would decrease by as much as 0.01 inches. This decrease in height was always accompanied by a small negative pore pressure which was measured at the base. As the pore fluid began to freeze the negative pressures increased and of course, the sample height again increased. The temperature effects on the sample height will affect the value of $\frac{S_t}{S_{\max}}$ to a greater or lesser extent depending on the overall strains involved in the particular thaw consolidation test.

The pressures measured in the thaw consolidation test are

quite accurate. The process of verifying the accuracy of the pressure measuring system at the start of each test series by carrying out a standard consolidation test ensured that the system was functioning correctly. It was found that the pressures measured were affected slightly by the temperature of the soil. As the temperature of the soil rose in the post thaw phase, the pressures measured would tend to rise. The rise in pressure counteracted the pressure drop due to dissipation to a small degree. The effects of temperature on the excess pore pressures obtained were never more than 1 to 2% of the total stress on the sample.

Every attempt was made to freeze the soil as quickly as possible and to do so under closed system conditions. The sample was frozen from the top and generally froze completely within thirty minutes after the freezing plane entered the top of the sample. There was undoubtedly some migration of water from the bottom of the sample to the top during the freezing process but the amount would be negligible due to the low permeability of the soil (1×10^{-7} cm/sec). By closing the lower porous stone to the atmosphere and freezing the sample at the top with dry ice, the soil could be completely frozen with virtually no change in overall moisture content. The void ratios which are shown in Table 1 of Appendices C to H for the frozen soils are, in most cases, 10% higher than the void ratio which existed before freeze commenced. This confirms that there was no substantial increase in overall water content as a result of the freezing process and that the increase in void ratio was due almost entirely to the phase change of water to ice.

The temperatures and step temperature increases were accurately maintained by the temperature controllers and no problems were encountered with this system. The recorded temperatures are accurate, but it proved difficult to determine precisely where the thermocouple or thermistor is measuring temperature. The thermistors on the side of the sample could easily be 0.05 inches higher or lower on the side of the sample. The thermocouple at the base measured consistently higher temperatures in test 5 - 6, than were expected. This thermocouple is thought to be defective.

Although no direct observations could be made as to the progress and shape of the thaw plane, it is possible to examine the available data to determine whether or not the thaw plane was planar and moved downwards at a rate proportional to the square root of time.

Examination of the settlement curves for tests 5 - 6 - 8 to 5 - 6 - 11 where 90% of the settlement was due to the phase change of ice to water, reveals that during thawing, settlement occurred proportional to the square root of time. Assuming the porosity of the soil is constant it is apparent from these curves that the thaw plane must have moved at a rate proportional to the square root of time.

The progress of the 32°F isotherm as it moves downward through the sample can be plotted from the temperature data. The data for test series 5 - 4 and 5 - 7 has been plotted in Fig. (5.5). Data for test series 5 - 6 is excluded because of the unreliable base temperature readings. The height of each thermistor has been normalized to the average height of the sample for the thaw phase of each test. In this case, since the temperature at the top of the sample was measured at

the top of the upper porous disc, the average height of the sample includes the thickness of the porous disc (.125 inches). It is assumed that the thaw plane moves through the porous disc at the same rate it moves through the soil. The value for the square root of time at which the 32° isotherm reached each thermistor has been normalized to the value for the square root of time when the 32° isotherm reached the base of the sample for each test. A plot of normalized depth versus normalized square root of time will, therefore, be independent of the α value for each test and the points should lie on the dotted line shown in Fig. (5.5) provided the 32° isotherm moves downwards proportional to the square root of time. Since the thermistors are located on opposite sides of the sample the points will be on the theoretical line only if the 32° isotherm moves downwards in a plane which is very nearly horizontal. The data plotted on Fig. (5.5) confirms that the 32° isotherm did move downwards in the permeode proportional to the square root of time. The scatter of the points is probably due to the fact that the height of the thermistors could not be precisely determined.

The base of the permeode should simulate the existence of an infinite depth of frozen soil below the sample itself. In order to check this condition, a computer program was run to determine the temperature isochrones for a semi-infinite soil mass subjected to the step increase in temperature which was used in the test. The solution employs a finite difference method formulated by Ho et al (1970) and was implemented on the computer by Mr. J. F. Nixon. The values for α and the moisture content of the soil as well as a typical unfrozen moisture content versus temperature curve for the soil were incorporated

into the solution. From the temperature isochrones derived in this manner it was possible to determine what the temperature at different times should be at a depth equal to the height of the soil sample. A theoretical base temperature versus time curve can thus be established. Such a curve is shown in Fig. (5.6). The actual points as measured by the thermocouple at the base of the soil for Test 5 - 4 - 6 have been plotted on this figure. The agreement is very close and is further evidence that the assumed heat boundary conditions were being correctly simulated in the permode. The base temperatures for all tests were checked in this way and very close agreement was found except for test series 5 - 6. Again, this is, in all probability, due to the faulty thermocouple used at the base for this test series.

The temperature data can be plotted in the form of temperature isochrones for any particular test. As an example, the temperature isochrones for test 5 - 4 - 4 have been plotted in Fig. (5.7). The shape of a typical isochrone determined from the computer solution previously described, has been plotted on the figure for comparison. The isochrones derived from the experimental results demonstrate a shape which is close to that predicted by the computer solution. The theoretical solution predicts a sharp break in the isochrone at the 32° isotherm as shown on the diagram. It is apparent from the experimental data that a sharp break does not occur, rather, that there is a smooth curve at this point. It is likely that this deviation is due to the fact that the thaw plane is in reality a zone of thaw rather than a planar surface.

The time that the thaw plane reaches the bottom of the sample

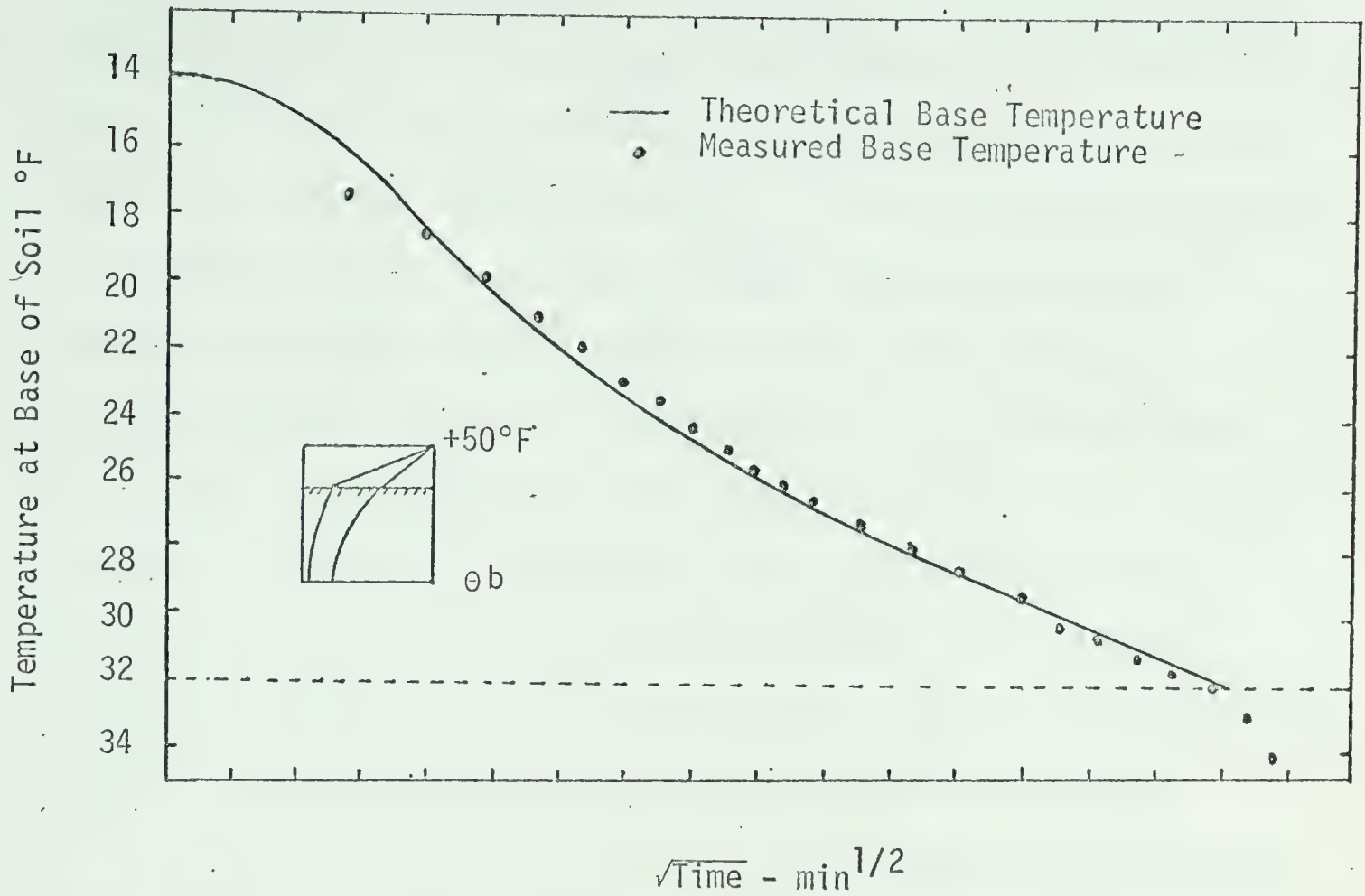


Fig. (5.6) Variation of Temperature at Base of Sample with Square Root of Time

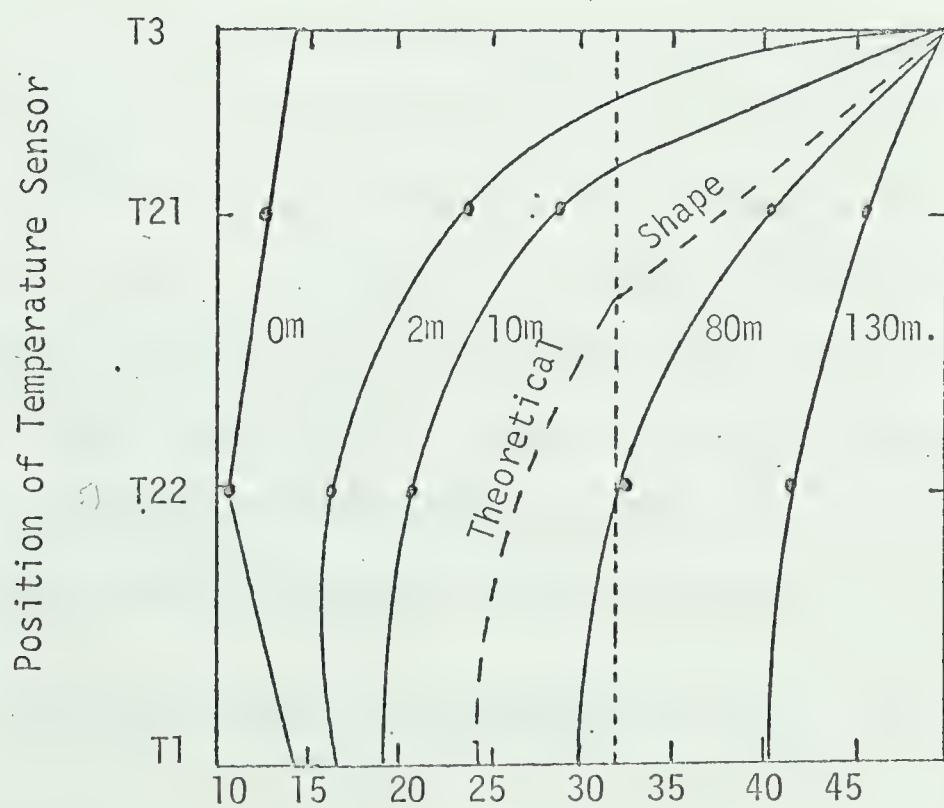


Fig. (5.7) Temperature Isochrones for Test 5-4-4

can be either well defined by the temperature, deflection and pressure data as for test 5 - 4 - 3, or very poorly defined as for test 5 - 4 - 6. The value for R is mildly sensitive to the selection of the time of end of thaw, however, the value for $\frac{S_t}{S_{\max}}$ is very sensitive to the height corresponding to the time of end of thaw. Some attempt was made to standardize the time selected as the end thaw time. In general the dial reading just prior to the maximum pore pressure reading seemed to give the most consistent results. In all cases the final selection for the time for end of thaw was made after considering the base temperature, pore pressure and deflection curve. The difficulty in pinpointing the time at which the thaw zone reaches the base of the soil is felt to be the major cause of the scatter of points on the $\frac{S_t}{S_{\max}}$ versus $\log R$ diagram. It is felt that the time the thaw plane reaches the base could be more accurately determined if two or three temperature sensors were placed at the base of the soil. This would provide a more definite indication of the exact time the thaw zone reaches the base and should improve the results obtained.

It will be noted that at the start of thawing, the slope of many of the settlement curves increases until a constant value is obtained after five or ten minutes. This is because several minutes are required to come up to the desired temperature on the top load cap. Until a constant temperature is reached the rate of thaw is always increasing. Furthermore, there is some water in the top porous stone which must melt before the thaw plane proceeds into the soil and this thaw would be accompanied by very little settlement.

It is possible to compare the experimental values for α with

values computed from the theoretical Neumann solution

where the value for α is given by:

$$\alpha = 2\lambda \sqrt{\kappa_2} \quad (5.6)$$

where κ_2 denoted the diffusivity of the thawed region,

and λ denotes the root of the following equation:

$$\frac{e^{-\lambda^2}}{\operatorname{erf}\lambda} - \left(\frac{k_1}{k_2}\right) \left(\frac{\kappa_2}{\kappa_1}\right)^{1/2} \left(\frac{T_1}{V-T_1}\right) \frac{e^{-\lambda^2 \kappa_1/\kappa_2}}{\operatorname{erfc}\left[\left(\frac{\kappa_2}{\kappa_1}\right)^{1/2} \lambda\right]} = \frac{\lambda L_s \pi^{1/2}}{C_2 (V - T_1)} \quad (5.7)$$

where the subscript '1' refers to the solid phase and the

subscript '2' refers to the liquid phase,

k denotes the conductivity,

κ denotes the diffusivity,

T_1 denotes the difference between the initial temperature

of the solid and the freezing temperature of the liquid,

V denotes the difference between the initial temperature of

the solid and the step temperature applied to the surface,

C denotes the volumetric specific heat,

L_s denotes the volumetric latent heat of fusion of the soil,

$\operatorname{erf}(x)$ denotes the error function,

and $\operatorname{erfc}(x) = 1 - \operatorname{erf}(x)$.

The various thermal properties for the soil can be determined from information given by Kersten (1940).

The specific heat is found from:

$$C_1 = \gamma_d (0.2 + 0.5 \omega) \quad (5.8)$$

and
$$C_2 = \gamma_d (0.2 + \omega) \quad (5.9)$$

where γ_d denotes the dry density of the soil,
and ω denotes the moisture content.

The conductivities K_1 and K_2 can be found for the soil from charts provided by Kersten.

The diffusivities κ_1 and κ_2 are found from

$$\kappa_1 = \frac{k_1}{C_1} \quad (5.10)$$

and

$$\kappa_2 = \frac{k_2}{C_2} \quad (5.11)$$

The volumetric latent heat of fusion for the soil can be determined from

$$L_s = L \omega \gamma_d \quad (5.12)$$

where L denotes the latent heat of fusion for water.

The initial temperature and step temperature are known and equation (5.7) can be solved for λ . A value for α can therefore be determined from equation (5.6).

The theoretical values for α were computed in this way for all the tests carried out on Athabasca clay. The results are presented in Table (5.1). The theoretical values for α have been plotted against the actual values obtained in Fig. (5.8). The figure demonstrates that there is excellent agreement between theoretical and experimental values. The scatter of the points could be due to experimental error in the determination of α or possibly due to the simplifying assumptions

TABLE 5.1

SUMMARY OF THERMAL CALCULATIONS

ATHABASCA CLAY

Test	e	w %	T _f °C	T _s °C	C ₁ cal/cc	C ₂ cal/cc	k ₁ cal/s °C cc x 10 ⁻³	k ₂ cal/s °C cc x 10 ⁻³	κ ₁ cm ² /s x 10 ⁻³	κ ₂ cm ² /s x 10 ⁻³	$\sqrt{\frac{\kappa_1}{\kappa_2}} A_1$	L _s	λ	$\frac{\alpha_{theor}}{\text{cm}^2/\text{sec}} \sqrt{1/2}$	$\frac{\alpha_{exp}}{\text{cm}^2/\text{sec}} \sqrt{1/2}$
5-4-3	1.03	38.8	-10	+10	0.52	0.775	5.2	2.75	10	3.55	0.595	41	0.24	0.0286	0.0259
5-4-4	0.904	34	-10	+10	0.52	0.76	5.0	3.1	9.6	4.1	0.654	38	0.244	0.0312	0.0245
5-4-5	0.808	30.8	-10	+10	0.523	0.75	4.9	3.2	9.3	4.28	0.68	36.2	0.249	0.0326	0.0243
5-4-6	0.704	26.4	-10	+10	0.524	0.73	4.8	3.5	9.2	4.8	0.724	32.1	0.258	0.0359	0.0262
5-5-3	1.401	53	-6.7	26.7	0.518	0.81	5.5	2.4	10.6	2.97	0.529	46.8	0.407	0.0443	0.0332
5-6-3	1.39	52.4	-9.5	33.4	0.518	0.81	5.5	2.4	10.6	2.97	0.529	46.6	0.436	0.0475	0.0459
5-6-4	1.038	39.2	-9.5	33.4	0.520	0.78	5.0	2.75	9.6	3.53	0.606	40.8	0.45	0.0535	0.0468
5-6-5	0.945	36.6	-6.7	31.7	0.52	0.77	5.0	2.9	9.6	3.76	0.626	39.6	0.46	0.0565	0.0563
5-6-6	0.822	31	-6.7	31.7	0.523	0.75	4.9	3.2	9.4	4.28	0.675	36.2	0.475	0.0621	0.0576
5-6-7	0.704	26.5	-9.5	10	0.524	0.73	4.8	3.5	9.2	4.8	0.722	33.3	0.258	0.0360	0.0222
5-6-8	0.677	25.5	-6.7	31.7	0.524	0.728	4.8	3.6	9.2	4.95	0.734	32.5	0.501	0.0707	0.0618
5-6-9	0.670	25.3	-6.7	29.5	0.524	0.728	4.8	3.6	9.2	4.95	0.734	32.5	0.475	0.067	0.0561
5-6-10	0.655	24.7	-6.7	10.6	0.525	0.723	4.8	3.7	9.2	5.12	0.75	31.9	0.288	0.0413	0.0379
5-6-11	0.654	24.6	-6.7	26.7	0.525	0.723	4.8	3.7	9.2	5.12	0.75	31.9	0.457	0.0653	0.0554
5-7-1	2.744	103.5	-6.7	40.6	0.51	0.88	5.5	1.7	10.8	1.94	0.424	58.6	0.462	0.0406	0.0594
5-7-2	1.014	36.2	-6.7	51.9	0.52	0.775	5.0	2.8	9.6	3.61	0.614	40.5	0.58	0.0696	0.0749

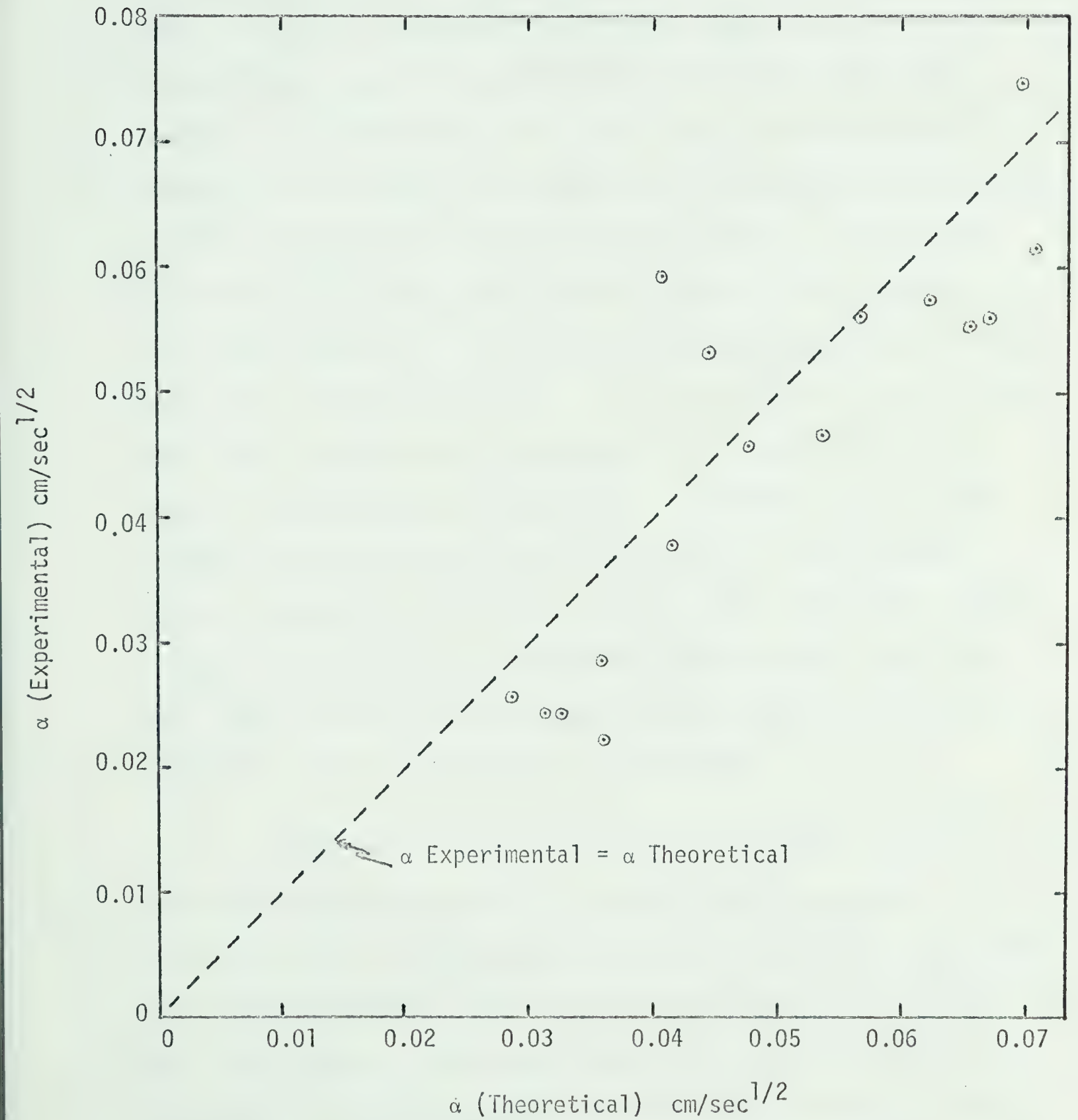


Fig. (5.8) Relationship between Rates of Thaw (α) determined Theoretically and Experimentally (Athabasca Clay)

that are made in determining the thermal properties for the soil in the theoretical solution.

The thaw consolidation ratio is calculated from the coefficient of consolidation which is determined from the post thaw settlement curve. If c_v has a substantially different value during the thaw portion of the test as compared to the measured post thaw value, then the value obtained for R will be in error. The variation of c_v with void ratio for the tests on Athabasca clay is shown on Fig. (5.4). It will be observed that for the majority of tests c_v varied from 0.5 to $3.0 \times 10^{-3} \text{ cm}^2/\text{sec}$ while the average void ratio ranged in value from 0.65 to 1.1. Fig. (5.4) shows that the greatest variation in c_v occur at lower void ratios. The pore pressures obtained from these tests indicate that R was correctly calculated using the c_v obtained from post thaw settlement data. Additional tests should be carried out to ascertain the behavior of c_v at very high void ratios. Indications are, however, that provided a very large change in void ratio does not occur during the thaw phase of a test, then the c_v obtained from the post thaw settlement curve will provide an essentially correct value for the overall test.

There is some error introduced when the final height of the sample is measured after it is removed from the permeometer. This error will affect the thaw-consolidation calculations in a negligible way and will cause the $e \log P$ curve to be shifted slightly up or down. The quality of the sample obtained by using the method of preparation previously described was found to be excellent. Visual examination showed the samples to be completely homogeneous throughout

and to contain no visible air voids. The final samples were found to be 100% saturated with the exception of the sample removed from the test 5-5. The latter was 94% saturated which may be due to the presence of air or to inaccuracy in the final height determination for this sample.

5.3 Recommendations for Improvements to Apparatus

The permode can be substantially improved in its design in order to facilitate its use and improve results. It is very difficult to set up the existing apparatus for tests on undisturbed frozen samples. The present pore pressure measuring system is prone to failure through leaks or plugging of the tubes. The outer steel jacket is not required for the low pressures encountered in testing. Provision should be made to measure deflection from a point located directly on the load cap.

One improved design which eliminates most of the above problems is shown in Fig. (5.9). The incorporation of a closed back-pressure consolidation cell into the permode has not been carried out in the new design. It is felt that the advantages to be gained through the use of back pressuring are outweighed by the increased complexity of the system. Back pressuring will be required in order to investigate the pressures generated during the freezing process because of the magnitude of the pressures involved. It would be preferable to develop a separate specialized piece of equipment to perform these investigations once the specific nature of the proposed experiments has been ascertained.

A closed back pressured system would permit the application of high loads on the sample through the use of air pressure. The loads required for most thaw-consolidation testing are less than 5.0kg/cm^2 . This pressure range can be easily attained with a direct loading system. The direct loading system also allows for

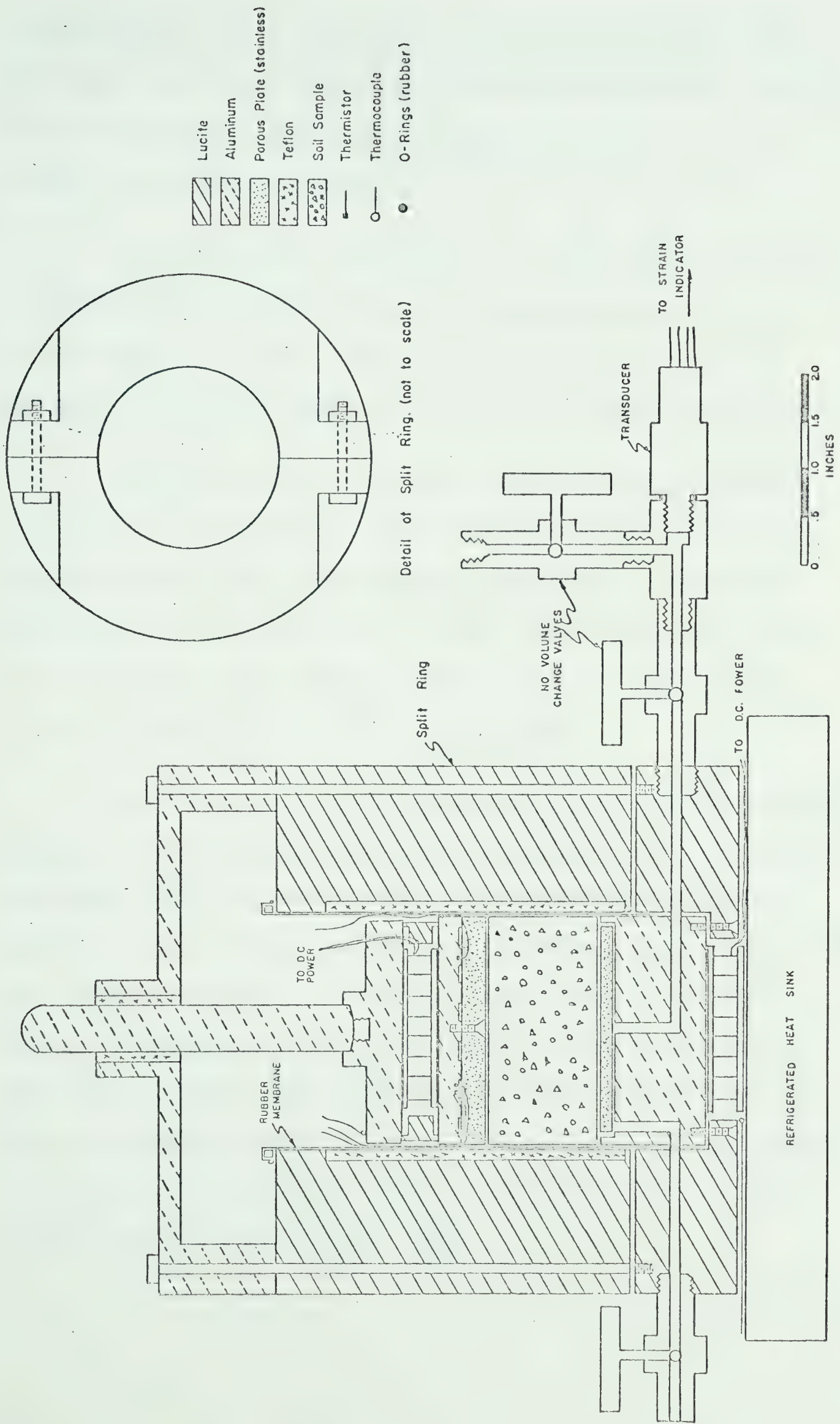


Fig. 5.9 Improved Permode Design

counterbalancing of the load hanger to allow for friction of the load cap. Furthermore, the pressure which can be applied to the load cap is limited by the compressive strength of the thermoelectric element (10kg/cm^2 on the sample).

A back pressured system permits the sample to be saturated to 100% by driving the air into solution under high pressure. It has been demonstrated that remolded samples can be placed in the permeometer to achieve very nearly 100% saturation without back pressure.

Undisturbed samples should be back pressured to simulate the pressures which existed in situ prior to testing. This is a great advantage of back pressuring but against this advantage must be weighed the disadvantage that it is no longer possible to quickly and conveniently freeze the sample from the top down by providing a heat sink such as dry ice above the load cap.

Improvements can also be made to the temperature controllers. A larger or better quality transistor should be provided to replace the present 2N5437 as this transistor is just barely adequate to handle the loads. The Variac permits the variation of the input current and should be retained. The feedback system should be modified so that it will automatically vary the polarity of the D.C. output as well as the current. Under the present system if a temperature of -10°C is selected and the heat sink is colder than -10°C , the controller will prevent the temperatures from rising above -10°C but will not prevent them from dropping below -10°C .

5.4 Conclusions

The data clearly demonstrates that within the framework of their assumptions, the physics of the thaw-consolidation problem have been correctly evaluated in the theory put forward by Morgenstern and Nixon (1971).

The permode proved capable of simulating the required temperature and stress boundary conditions for one-dimensional thaw consolidation.

The validity of applying the Neumann solution to predict the movement of the thaw plane for the one-dimensional case has been confirmed.

The fundamental importance of the thaw consolidation ratio as a parameter indicating the pore pressures developed at the thaw plane has been demonstrated. The rate of settlement during thaw and after thawing has stopped has also been predicted correctly by the theory. The degree of consolidation which occurs during thawing was shown to be dependent on R , however, there was some scatter of the experimental points. This is primarily due to inaccuracies in measuring deflection and the difficulty in determining the exact time that the thaw plane reached the base of the sample.

The thaw consolidation ratio R will provide a useful field guide in determining which soils will become unstable under given applied pressures and thermal loadings. Field evidence (Brown and Johnson, 1970) suggest that in many instances thawing does occur almost

proportional to the square root of time. Where thawing does not occur proportional to the square root of time, it can be shown that the parameter R is not independent of time. Investigations should therefore be carried out to determine whether or not the assumption of thawing proportional to the square root of time will cause significant differences in predicted pore pressures and settlements when other rates of thaw occur.

The pore pressures obtained were generally slightly higher than the pressures predicted by the theory. Indications are that this effect is due to non-linearity of the $e - \sigma'$ relationship. If this is the case, then the effect would be more pronounced for soils at high initial void ratios. Further investigation is required in this area.

The assumption that c_v , for the overall test, can be determined from the post thaw settlement curve appears to be acceptable for the range of void ratios tested. Investigations should be carried out to determine the behavior of c_v at high void ratios and hence determine the validity of this assumption for soils at high initial void ratios.

The initial effective stress (σ_0') was assumed to be zero for the tests conducted and the results indicate that this assumption was essentially valid. It is recognized, however, that under certain circumstances the initial effective stress could attain a significantly high value and the pore pressures obtained would be much less than the predicted values. An investigation into the degree of overconsolidation which occurs as a result of the freezing process should be made

and the results obtained related to the pressures which occur on thaw. Such an investigation would serve to further the present understanding of the whole freeze-thaw phenomenon as related to porous media.

LIST OF REFERENCES

- Barden, L., and Berry, P.L., 1965, "Consolidation of normally consolidated clay". J. Soil Mechs. and Found. Div. Proc. ASCE, SM5, 15-35.
- Brown, W.G. and Johnston, G.H., 1970, "Dikes on permafrost: Predicting thaw and settlement". Canadian Geotechnical Journal, 7, 365-371.
- Carslaw, H.S. and Jaeger, J.C., 1947, "Conduction of heat in solids". Claredon Press, Oxford.
- Chattopadhyay, P.K., 1969, "Consolidation modification by sodium chloride additive". M.E. Sc. Unpublished thesis, University of Western Ontario.
- Davis, E.H. and Raymond, G.P., 1965, "A non linear theory of consolidation". Geotechnique, 15, 161-173.
- Ho, D.M., Harr, M.E. and Leonards, G.A., 1970, "Transient temperature distribution in insulated pavements: predictions versus observations". Canadian Geotechnical Journal, 7, 275-284.
- Kersten, M.S., 1949, "Laboratory research for the determination of the thermal properties of soils". Minnesota University, Minnesota. Published by National Technical Information Service, U.S. Dept. of Commerce.
- Lambe, T.W., 1951, "Soil testing for engineers". John Wiley & Sons, Inc., New York, 60.
- Leonards, G.A. and Girault, P., 1961, "A study of the one-dimensional consolidation test". Proc. 5th Int. Conf. Soil Mechs. Found. Eng., 1, 213.

- Morgenstern, N.R. and Nixon, J.F., 1971, "One dimensional consolidation of thawing soils". Canadian Geotechnical Journal, 8, 558-565.
- Perloff, W.H., Nair, K., and Smith, J.G., 1965, "The effect of measuring system on pore water pressures in the consolidation test", Proc. 6th Int. Conf. Soil Mechs. Found. Eng., 1, 338.
- Taylor, D.W., 1948, "Fundamentals of Soil mechanics". John Wiley & Sons, Inc., New York.
- Tsytoich, N.A., Zaretskii, Y.K., Gregoryeva, V.G., Ter-Martirosyan, Z.G., 1965, "Consolidation of thawing soils". Proc. 6th Int. Conf. Soil Mechs. Found. Eng., 1, 390-394.
- Williams, P.J., 1967, "Properties and behavior of freezing soils". Norwegian Geotechnical Inst. Publication No. 72.
- Zaretskii, Y.K., 1968, "Calculation of the settlement of thawing soil". Soil Mechs. and Found. Eng. (May-June), No. 3, 151-155.

ADDITIONAL REFERENCES

Freeze Thaw Phenomenon in Soil

- Aldrich, H.P., 1956, "Frost Penetration Below Highway and Airfield Pavements". Highway Research Board Bulletin 135 (1956) 124-144.
- Beskow, G., 1935, "Soil Freezing and Frost Heaving With Special Application to Roads and Railroads". Swedish Geotechnical Society, Series C, No. 375, Translated by J.O. Osterberg, Northwestern Univ., No. 1947.
- Bolduzzi, F., 1960, "Experimental Investigation of Soil Freezing", National Research Council Publication, TT912.
- Broms, Bengt. B., and Yao. Y.C., 1964, "Shear Strength of a Soil After Freezing and Thawing". Jour. S.M.F. Proc. ASCE, July 1964.
- Casagrande, A., 1931, "Discussion on a New Theory of Frost Heaving", by A.C. Benkelman and F.R. Olmstead, Proc. HRB 11, Part 1: 168-72.
- Chahal, R.S. and Yong. R.N., 1965, "Validity of Soil Water Characteristics Determined with the Pressurized Apparatus Soil Science", 99, 2: 98-103.
- Csathy, T.I. and Townsend. D.L., 1962, "Pore Size and Field Frost Performance of Soils, HRB Research Bulletin 33: 67-80.
- Dillon, H.B. and Andersland, O.B., 1966, "Predicting Unfrozen Water Contents in Frozen Soils". Canadian Geotechnical Journal, 3, 53-60.
- Everett, D.H., 1961, "The Thermodynamics of Frost Action in Porous Soils", Trans. Faraday Soc. 57: 1541-51.

- Grim, Ralph E. "Applied Clay Mineralogy", McGraw Hill Publication.
- Hoekstra, P., Chamberlain. E., and Frate. A., 1965. "Frost Heaving Pressures", HRB Record 101: 28-38.
- Janson, L.E., 1964, "Frost Penetration in Sandy Soil", Royal Institute of Technology No. 231.
- Kersten, M.S., 1948, "Specific Heat Tests on Soils". Proc. 2nd Int. Conf. on Soil Mechs. and Found. Eng., 3, 12-15.
- Keune, R. and Hoestra, P., 1967, "Calculationg the Amount of Unfrozen Water in Frozen Ground from Moisture Characteristic Curves", Cold Regions Research and Experimental Laboratory Special Report 114.
- Lovell, C.W., 1957, "Temperature Efferts on Phase Composition and Strength of Partially Frozen Soils" Highway Research Board, Bulletin 168, 74-95.
- Malyshev .A.V., 1966, "Calculation of the Settlement of Foundations on Thawing Soils", Soil Mechanics and Foundation Engineering, July-August, 260.
- McKyes, S.E., 1966, "Theoretical and Experimental Determination of Frost Heaving Pressure in Partially Frozen Silt", B.Eng. (Hons.) Thesis, Dept. of Civil Eng., McGill Univ.
- Miller, R.D., Baker, J.H., and Kolaian. J.H., 1960, "Particle Size, Overburden Pressure, Pore Water Pressure, and Freezing Temperature of Ice Lenses in Soil", Trans. 7th Int. Cong. of Soil Science 1:122-8.
- Osler, J.C., 1966, "Studies of the Engineering Properties of Frozen Soils", Report No. D. Phys. R(G) Misc. 25, Directorate of Physical Research, DRB, Canada.

- Osler, J.C., 1967, "The Influence of Depth of Frost Penetration on the Frost Susceptibility of Soils", Canadian Geotech. Journal Vol. IV, No. 3, August 1967, 334-346.
- Palmer, A.C., 1967, "Ice Lensing, Thermal Diffusion and Water Migration in Freezing Soil", J. of Glaciology, 6, No. 47.
- Penner, E., 1958, "Pressure Developed in a Porous Granular System as a Result of Ice Segregation", HRB Spec. Rept. 40:191-9.
- Penner, E., 1957, "Soil Moisture Tension and Ice Segregation", HRB Bulletin 68:50-64.
- Penner, E., 1959, "The Mechanism of Frost Heaving in Soils", HRB Bulletin 225: 1-13.
- Penner, E., 1962, "Thermal Conductivity of Saturated Leda Clay", Geotechnique 12, 168-175.
- Penner, E., 1966, "Pressure Developed During the Unidirectional Freezing of Water-Saturated Porous Materials, Experiment and Theory", presented to Intern. Conf. on Low Temp. Sci., Sapporo, Japan.
- Penner, E., 1967, "Heaving Pressure in Soils During Unidirectional Freezing", Canadian Geotechnical Journal, Vol. IV, No. 4, Nov. 1967, 398-408.
- Penner, E., 1970, "Thermal Conductivity of Frozen Soils", Can. J. of Earth Sciences, 7, 982.
- Sangers, Frederick J., 1968, "Ground Freezing in Construction", Journal S.M.F. ASCE, Jan. 1968.
- Skaven-Haug, O.V., 1959-60, "Protection Against Frost Heaving on the Norwegian Railways", Geotechnique, Vol. 9-10, 1959-60, 87.
- Stuart, J.G., "Consolidation Tests on Clays Subjected to Freezing and

Thawing", Swedish Geot. Inst., Reprints & Preliminary Reports No 7. 1-9.

Tsyтовish, N.A., 1957, "The Fundamentals of Frozen Ground Mechanics (New Investigations), Proc. 4th Int. Conf. Soil Mechs. and Found. Eng., 1, 116.

Tsyтовich, N.A., and Khakimov, Kh.R., 1961, "Ground Freezing Applied to Mining and Construction", Proc. 5th Int. Conf. Paris 1961.

Tsyтовich, N.A., 1965, "Permafrost in the U.S.S.R. as Foundations for Structures", Proc. 6th Int. Conf. Soil Mechs. and Found. Eng., 1, 360.

Yong, R.N. and Warkentin, B.P, "Introduction to Soil Behaviour", Macmillian Co., N.Y.

ADDITIONAL REFERENCES

Consolidation and Pore Pressure Measurement

- Abbott, M.B., 1960, "One Dimensional Consolidation of Multi-Layered Soils", *Geotechnique*, 10.
- Bishop, A.W., and Henkel, B.J., "The Measurement of Soil Properties in the Triaxial Test", Arnold and Co., London, 183-208.
- Fox, E.N., 1948, "The Mathematical Solution for the Early Stages of Consolidation", *Proc. 2nd Int. Conf. of Soil Mechs. and Found. Eng.*, 1.
- Gibson, R.E. and Lumb, P., 1953, "Numerical Solution of Some Problems in Consolidation of Clay". *Proc. Inst. of Civil Eng.*, 1.
- Gibson, R.E., 1958, "The Progress of Consolidation in a Clay Layer Increasing in Thickness with Time", *Geotechnique*, 8, 171-182.
- Gibson, R.E., England, G.L., Hussey, M.J.L., 1967, "The Theory of One Dimensional Consolidation of Saturated Clays", *Geotechnique*, 17, 261-273.
- Hanshaw, B.B., and Bredehoeft, J.D., 1968, "On the Maintenance of Anomalous Fluid Pressures: II Source Layer at Depth", *Geol. Soc. of America, Bulletin* V79, 1107-1122.
- Northey, R.D., and Thomas, R.F., 1965, "Consolidation Test Pore Pressures" *Proc. 6th Int. Conf. of Soil Mechs. and Found. Eng.*, 1, 323.
- Poskitt, T.J., and Birdsall, R.O., 1971, "A Theoretical and Experimental Investigation of Mildly Nonlinear Consolidation Behavior in Saturated Soil", *Can. Geot. J.*, 8, 182-216.

Wissa, E.Z., and Heiberg, S., 1969, "A New one-dimensional Consolidation Test", Mass. Inst. of Tech., Dept. of Civil Eng. Publication.

APPENDIX A

SUMMARY OF SOIL PROPERTIES

ATHABASCA CLAY

LAKE EDMONTON CLAY

BENTONITE

ATHABASCA CLAY

Unified Soil Classification CL

Liquid Limit	40.8
Plastic Limit	20.3
Plasticity Index	20.5
G_s	2.65

Approximate Consolidation Parameters (No Freeze Cycle)

$$c_v = 1 \times 10^{-4} \text{ to } 1 \times 10^{-3} \text{ cm}^2/\text{sec}$$

$$k = 1 \times 10^{-8} \text{ to } 1 \times 10^{-7} \text{ cm/sec}$$

Grain Size Curve Attached, See Fig. (A.1).

LAKE EDMONTON CLAY*

Unified Soil Classification CH

Liquid Limit	70.0
Plastic Limit	29.0
Plasticity Index	41.0
G_s	2.76

Approximate Consolidation Parameters (No Freeze Cycle)

$$c_v = 1.7 \times 10^{-4} \text{ cm}^2/\text{sec}$$

$$k = 10^{-6} \text{ to } 10^{-8} \text{ cm/sec}$$

Grain Size Curve Attached, See Fig. (A.2)

*Source

Thomson, S., 1969, "A Summary of Laboratory Results on Lake Edmonton Clay" Unpublished Internal Note: SM2, University of Alberta.

BENTONITE*

Unified Soil Classification CH

Liquid Limit 591.

Plastic Limit 87.

Plasticity Index 504.

 G_s 2.75

Approximate Consolidation Parameters (No Freeze Cycle)

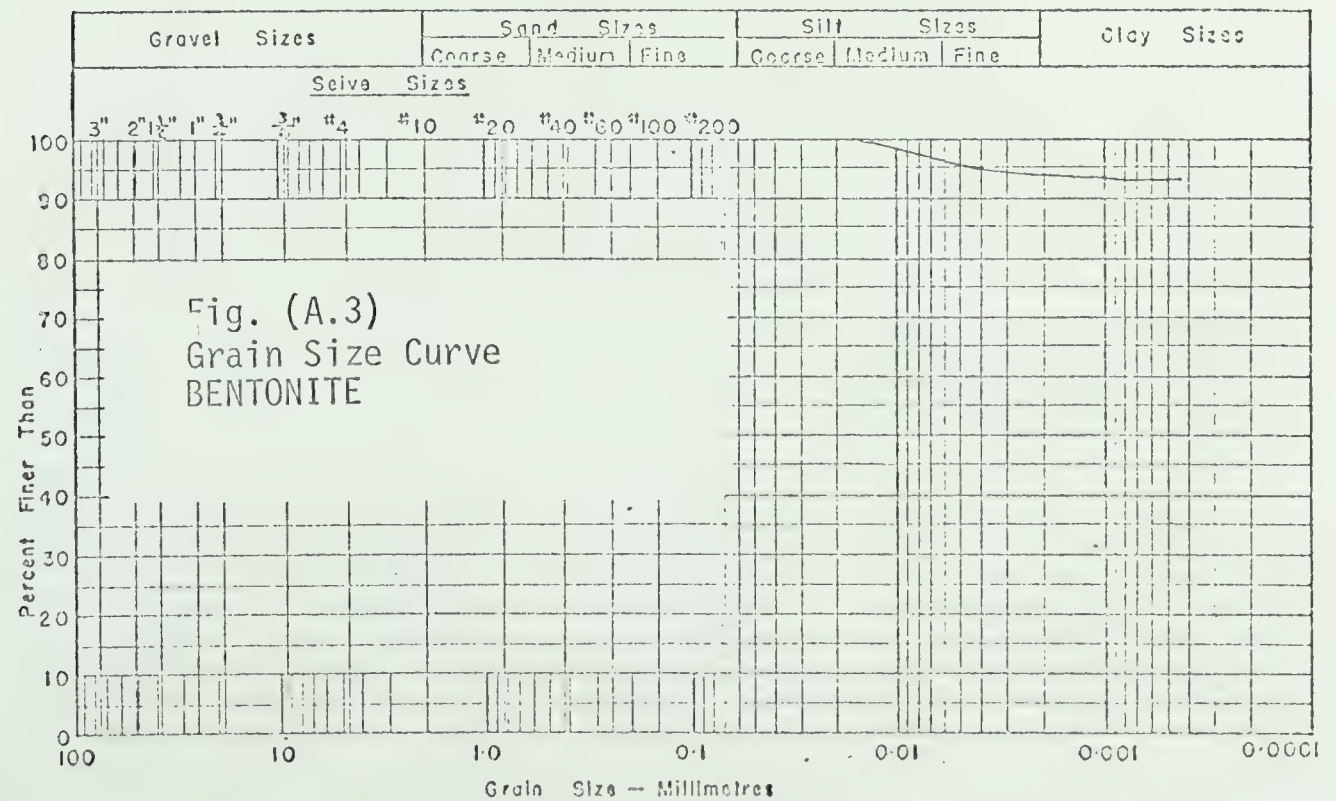
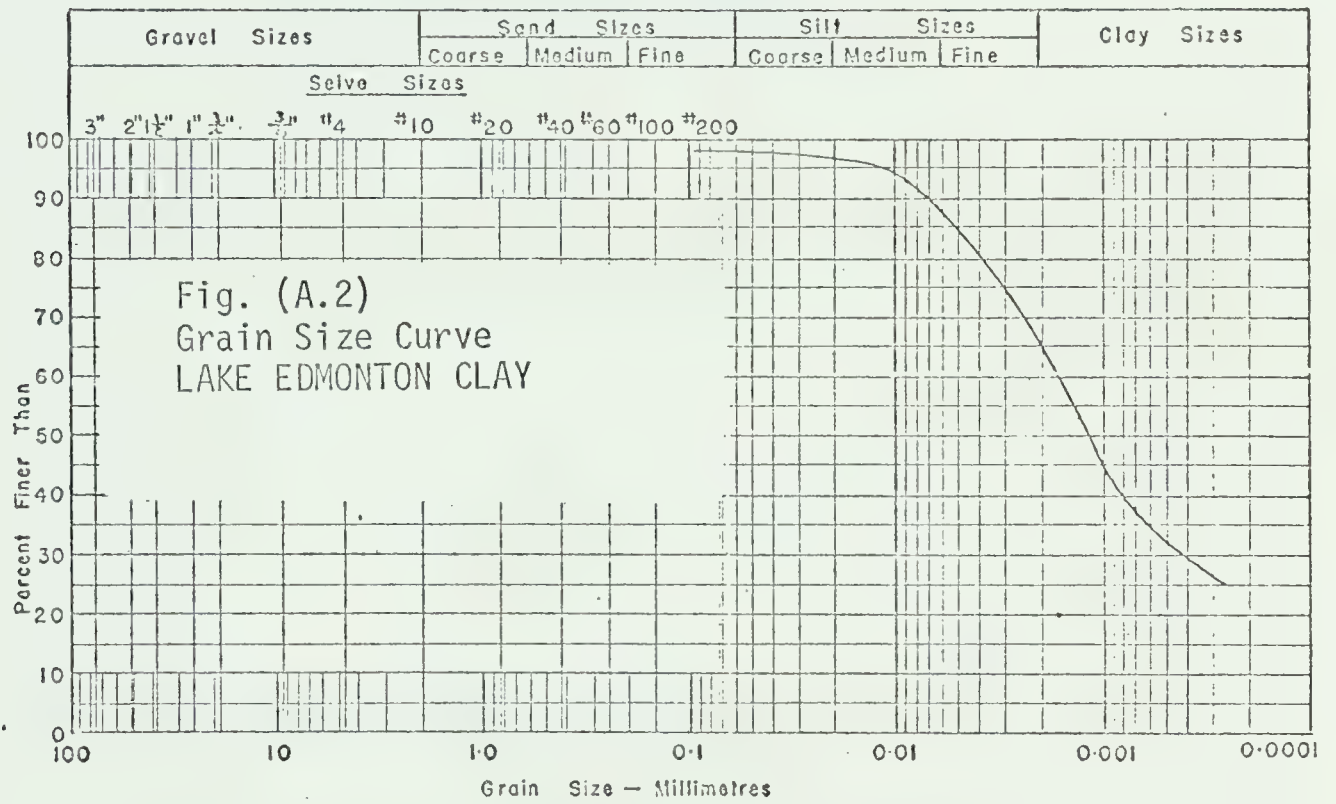
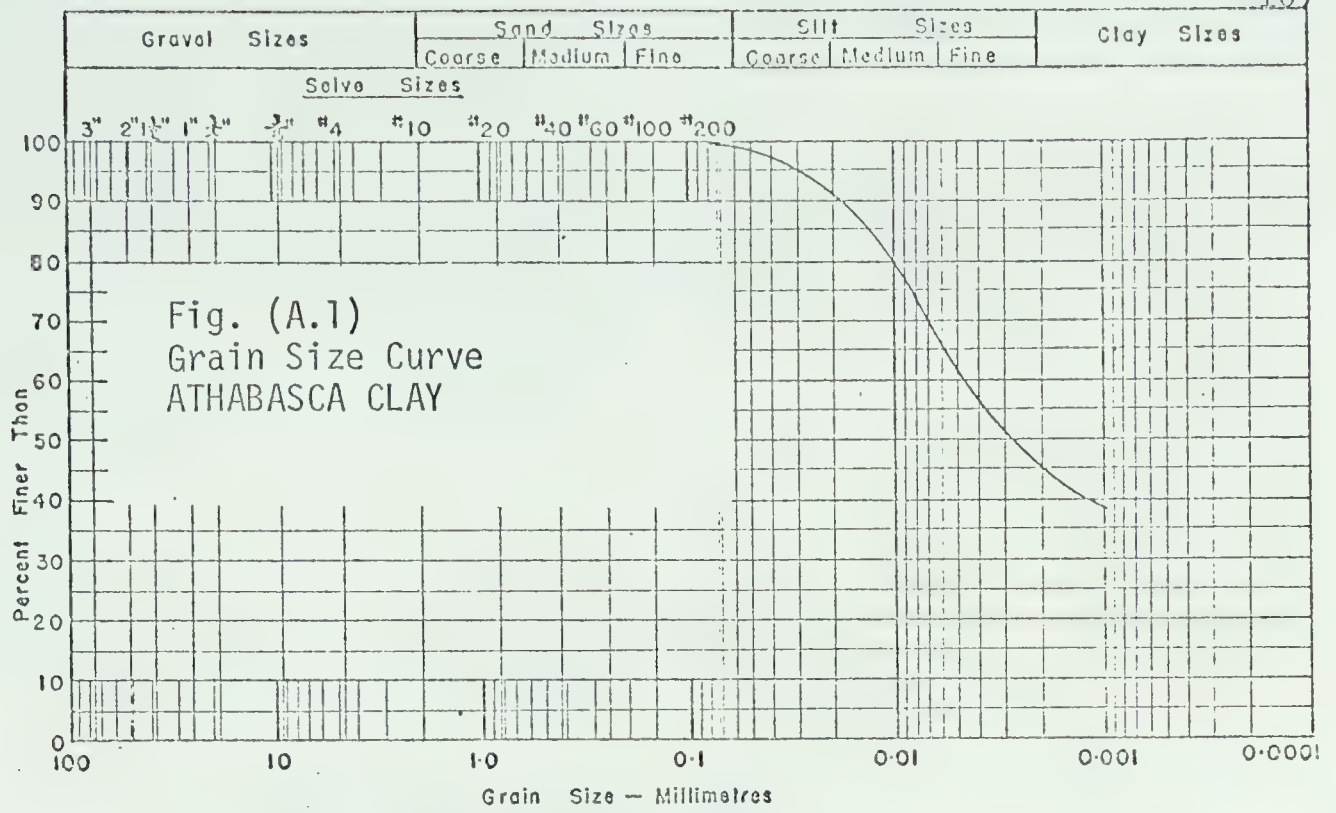
$$c_v = 0.7 \times 10^{-5} \text{ cm}^2/\text{sec}$$

$$k = 1 \times 10^{-9} \text{ cm/sec}$$

Grain Size Curve Attached, See Fig. (A.3).

* Source

Ganapathy, G.V., 1969, "Some Fundamental Aspects of the Residual Strength of Clays", Unpublished M. Eng. Report, University of Alberta.



APPENDIX B

SPECIFICATIONS OF MAJOR COMPONENTS OF TEMPERATURE CONTROL DEVICES

THERMOELECTRIC ELEMENTS

MC5-8-12 Pre-insulated surface, Thermo-electric modules.

Available from Materials Electronic Products Corporation

990 Spruce Street, Trenton, N.J. 08638

THERMISTORS

YSI 511 Non-interchangeable thermistor probes.

Available from Yellow Springs Instrument Co., Incorporated,

Yellow Springs, Ohio.

TEMPERATURE RECORDER

Honeywell Electronik 15 Strip Chart Multipoint

Recorder 15303836-24-04-0-000-004 10

Available from Honeywell Industrial Products Group,

Wayne and Windrim Avenue, Philadelphia 44, Pa.

TEMPERATURE CONTROLLER - MAJOR COMPONENTS

VARIAC

Type W5MT3A Autotransformer

Available from General Radio Company Concord, Mass.

Hammond 60HZ Filament Transformers, Model 167V12

Potentiometers, Model IRC 8400, 20KOHMS

RD 412 IRC Dials

Available from Cardinal Industrial Electronics Ltd.,

14310 - 118th Avenue, Edmonton, Alberta

Transistors, Germanium Power, PNP, Type Mota ZN5437

Transistors, Silicon Power, NPN, Type Mota ZN5067

Transistors, Silicon Power, PNP, Type Mota ZN5679

Available from Bowtek Electric, Vancouver, B.C.

APPENDIX C

SUMMARY OF DATA

Test Series 5 - 4

SUMMARY OF RESULTS
TEST SERIES 5-4

SOIL - ATHABASCA CLAY

Test	Phase	Press kg/cm ²	Dial in	H in	e	t ₅₀ min	k cm/sec x 10 ⁻⁷	c _v cm ² /gm x 10 ⁻³	m _v cm ² /gm x 10 ⁻³	√Time (End Thaw) Min ^{1/2}	X (in)	α cm/sec ^{1/2}	R = $\frac{\alpha}{2\sqrt{c_v}}$	$\frac{S_t}{S_{max}}$	$\frac{U_n}{p_o}$
5-4-0	Start	.0845	.7640	1.2737	2.170	-	-	-	-	-	-	-	-	-	-
	End		.4700	.9797	1.447										
5-4-1	Start	.179	.4700	.9797	1.447	75	2.3	0.25	.92	-	-	-	-	-	-
	End		.3987	.9084	1.265										
5-4-2	Start	.179	.3987	.9084	1.265										
	Frozen	.179	.4469	.9084	1.265										
	End Thaw		.3452	.8549	1.130	22	2.55	0.95	.268	13.9	.9057	.0214	.347	$\frac{.1535}{.0935} = .57$	.61
	End		.3052	.8149	1.030										
5-4-3	Start	.179	.3052	.8149	1.030										
	Frozen	.367	.3455	.8552	1.130										
	End Thaw		.2850	.7947	0.980	13.5	1.45	1.35	.107	10.45	.8249	.0257	.354	$\frac{.0202}{.0514} = .39$	.540
	End		.2538	.7635	0.904										
5-4-4	Start	.367	.2538	.7635	0.904										
	Frozen	.367	.2978	.8075	1.010										
	End Thaw		.2472	.7569	0.890	10	0.74	1.699	.044	10.49	.7822	.0245	.299	$\frac{.0066}{.0177} = .37$	.191
	End		.2361	.7458	0.860										
5-4-5A	Start	.745	.2361	.7458	0.860	10	0.64	1.637	.050	-	-	-	-	-	-
	End		.2194	.7291	0.794										
5-4-5	Start	.745	.2194	.7291	0.794										
	Frozen	1.525	.2514	.7611	0.898										
	End Thaw		.1924	.7021	0.751	16	0.16	0.902	.018	9.8	.7266	.0243	.407	$\frac{.0273}{.0463} = .59$	.317
	End		.1734	.6831	0.704										
5-4-6	Start	1.525	.1734	.6831	0.704										
	Frozen	2.220	.2037	.7134	0.780										
	End Thaw		.1632	.6729	0.676	21	0.04	0.639	.006	7.9	.6931	.0288	.569	$\frac{.0102}{.0200} = .51$	.152
	End		.1534	.6631	0.654										

END OF TEST DATA

ω = 25.0%
W_s = 86.33gm
Final ht = .6631 in
H_o = .4010 in
S = 101.5%

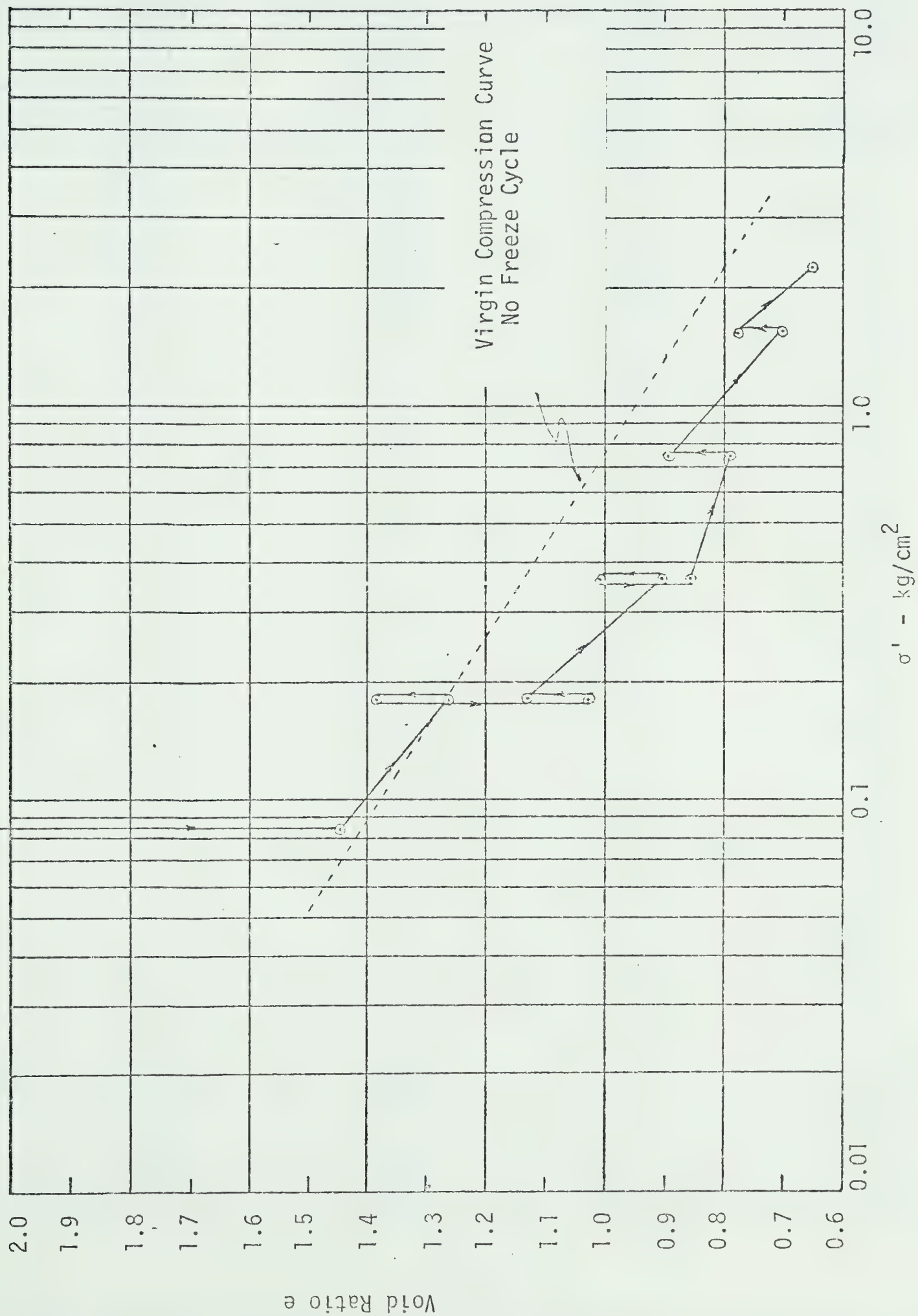


Fig. (C.1) Void Ratio versus Effective Stress Test Series 5-4

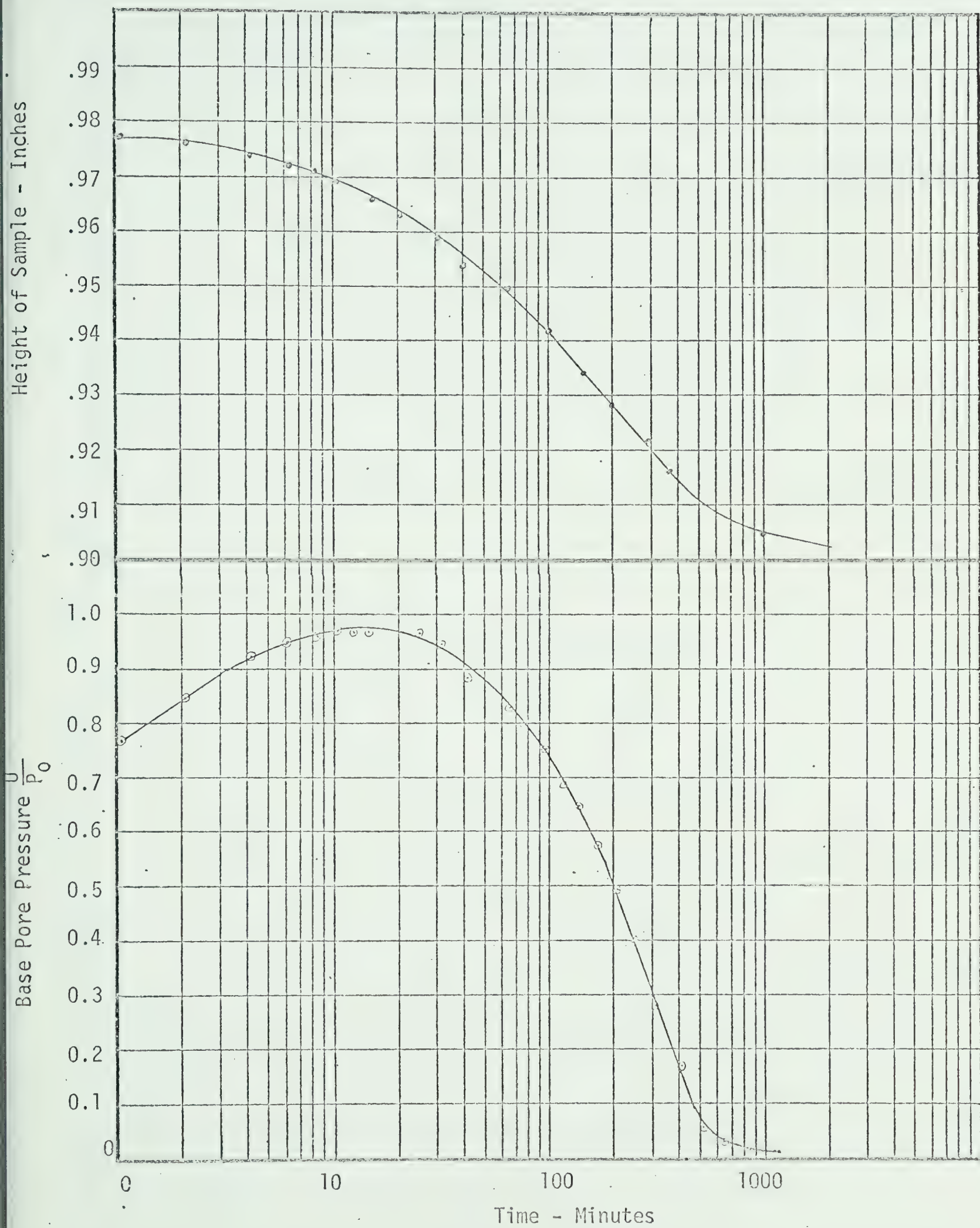


Fig. (C.2) Settlement and Pore Pressure Dissipation Curves Test 5-4-1

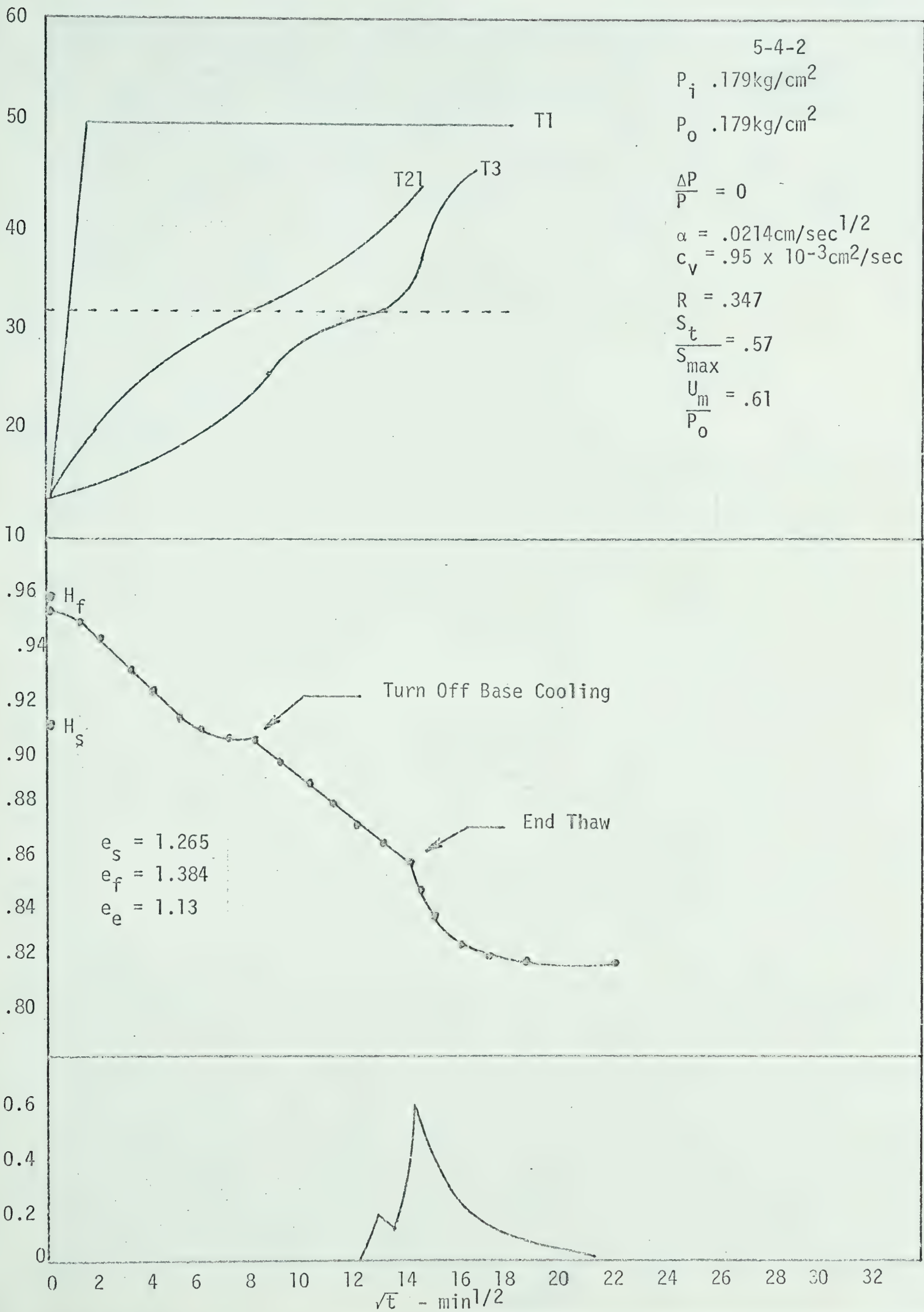
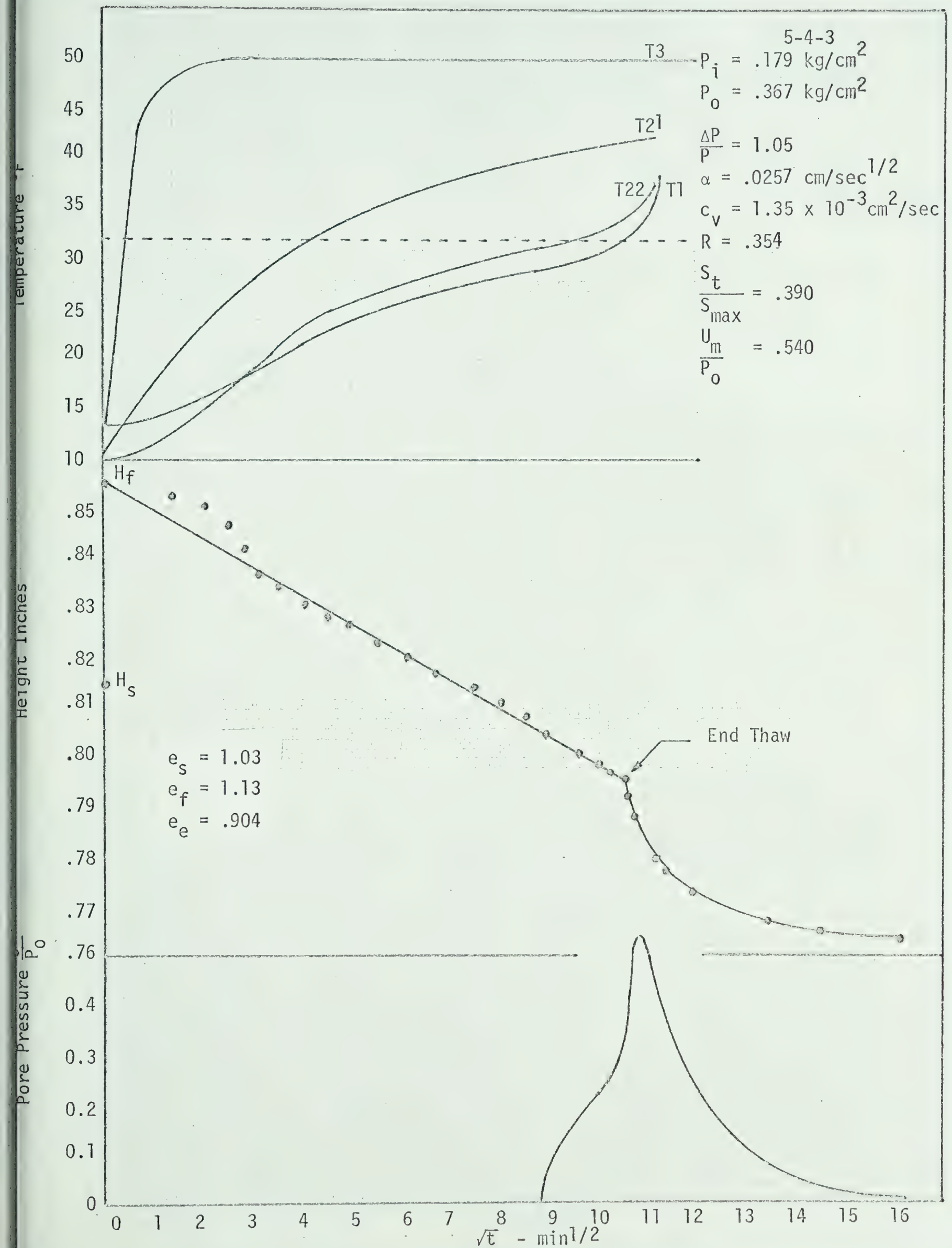
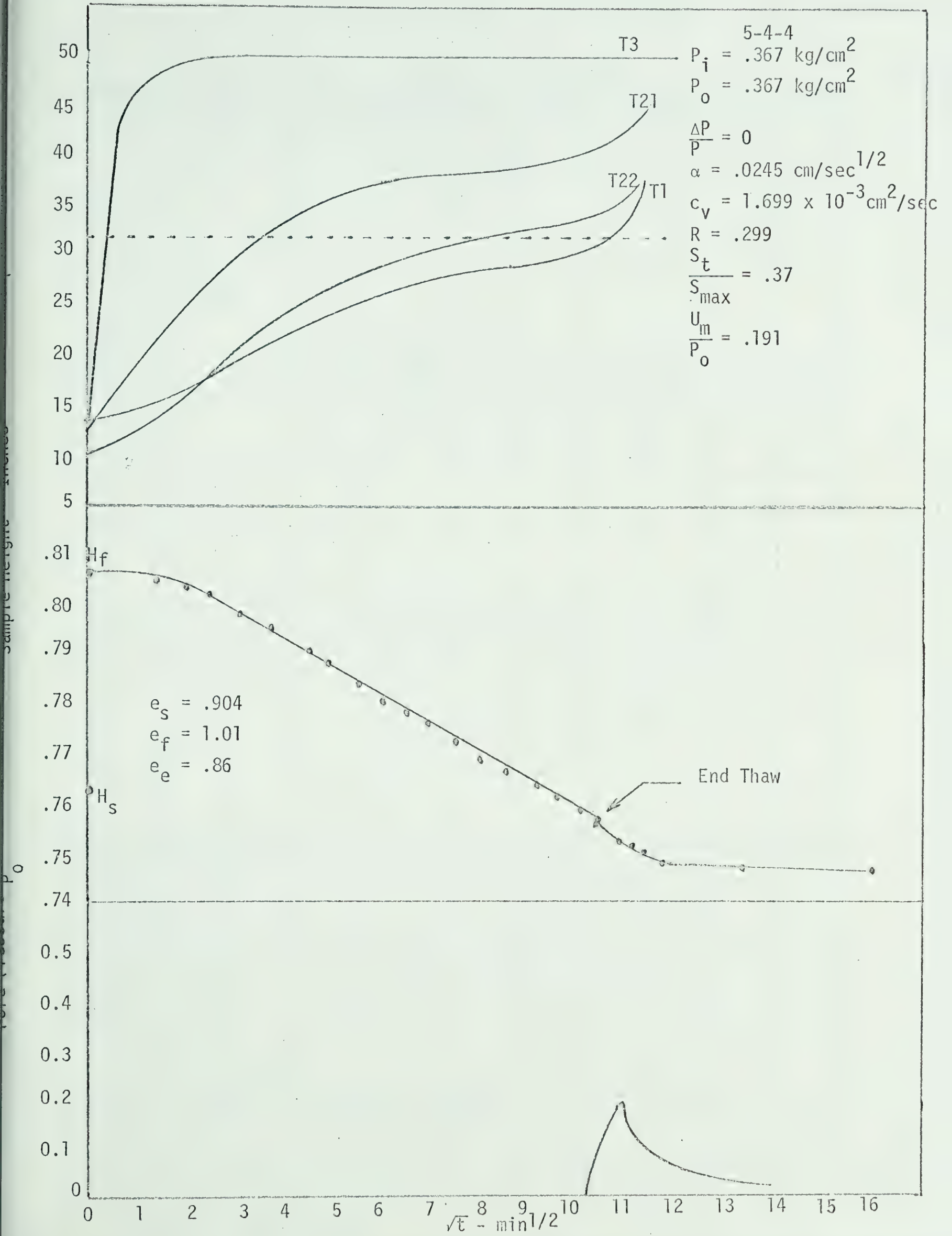


Fig. (C.3)





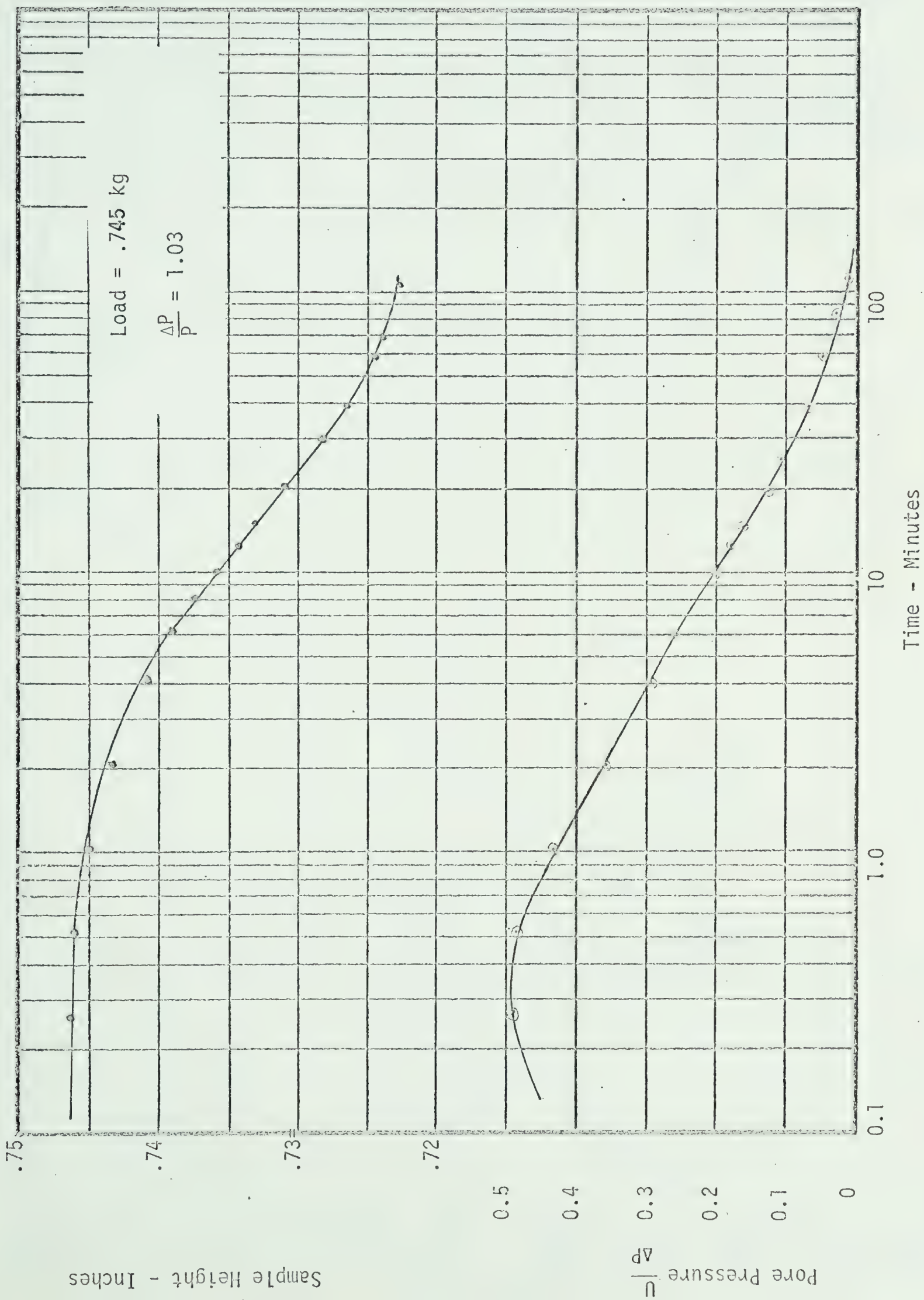


Fig. (C.6) Settlement and Pore Pressure Curves - Test 5-4-5A

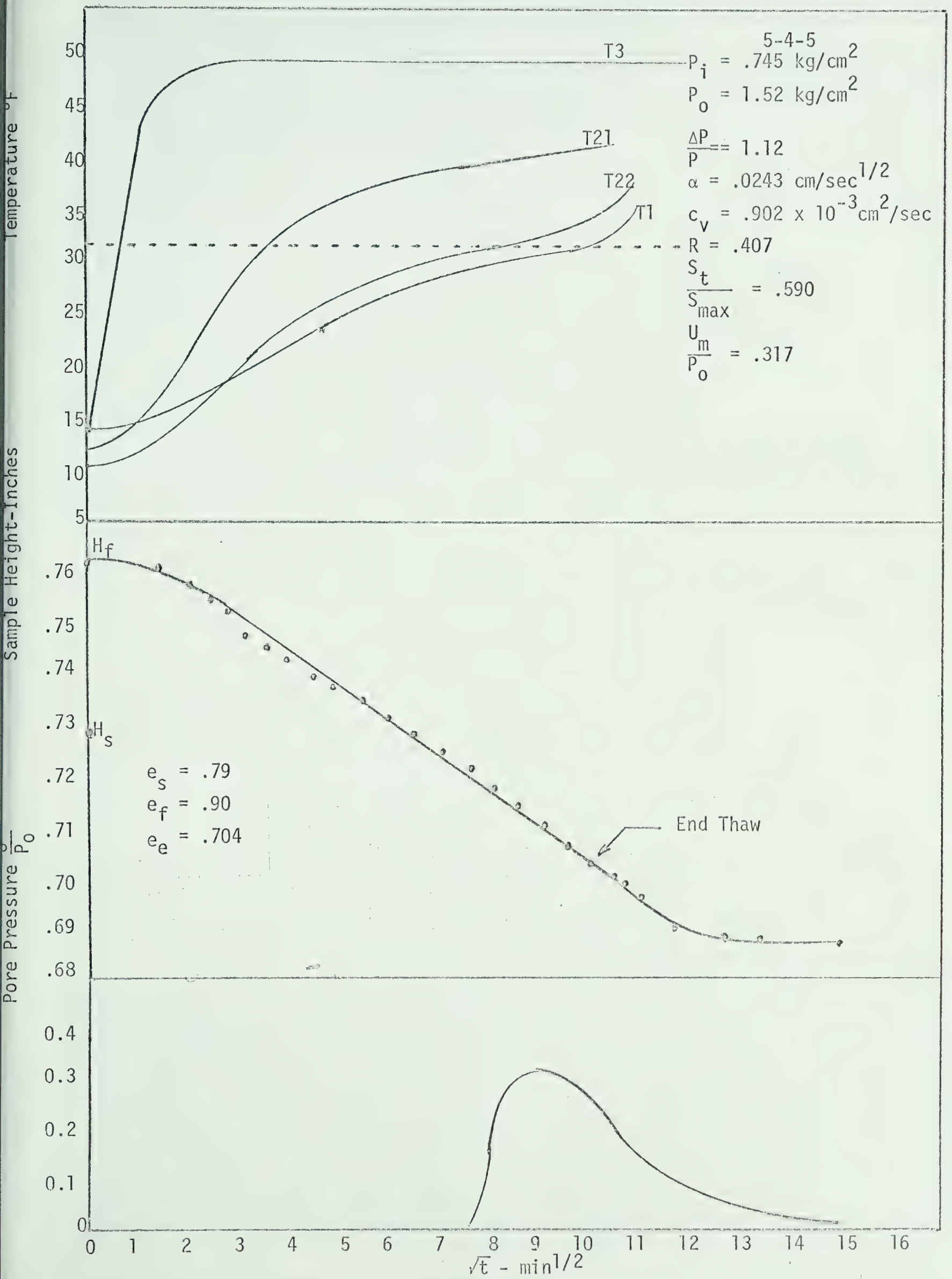


Fig. (C.7)

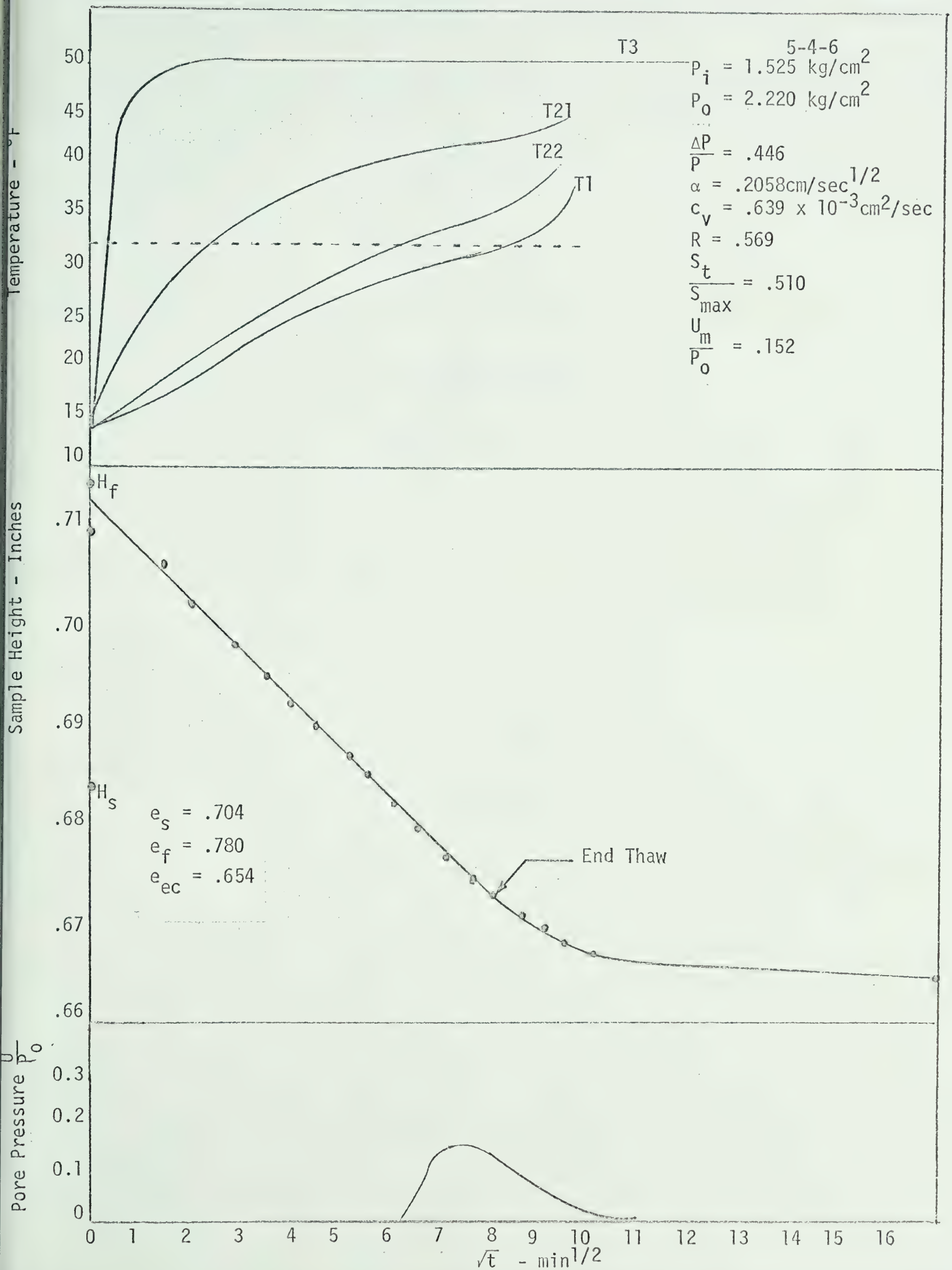


Fig. (C.8)

APPENDIX D

SUMMARY OF DATA

Test Series 5 - 5

APPENDIX D - TABLE 1

SUMMARY OF RESULTS
TEST SERIES 5-5

SOIL - ATHABASCA CLAY

Test	Phase	Press kg/cm ²	Dial in	H in	e	t ₅₀ min	k cm/sec x 10 ⁻⁷	c _v cm ² /sec x 10 ⁻³	m _v cm ² /gm x 10 ⁻³	$\sqrt{\text{Time}}$ (End Thaw) Min ^{1/2}	X (in)	$\frac{\alpha}{\text{cm/sec}^{1/2}}$	R = $\frac{\alpha}{2\sqrt{c_v}}$	$\frac{S_t}{S_{\text{max}}}$	$\frac{U_m}{P_0}$
5-5-1	Start	.06	.5000	1.1394	2.165	-	-	-	-	-	-	-	-	-	-
	End		.2777	.9171	1.549										
5-5-2	Start	.125	.2777	.9171	1.549	100	1.5	.167	0.9	-	-	-	-	-	-
	End		.2250	.8644	1.401										
5-5-3	Start	.125	.2250	.8644	1.401										
	Freeze	.250	.2584	.8978	1.493										
	End Thaw		.1330	.7724	1.140	13	.42	1.28	0.33	5.15	.8351	.0532	.729	$\frac{.0920}{.1504}$	.738
	End		.0746	.7140	0.972										

END OF TEST DATA

$\bar{w} = 34.5\%$
 $w_s = 77.40\text{gm}$
 Final ht = .7140 in
 $H_0 = .3600$ in
 $S = 94.0\%$

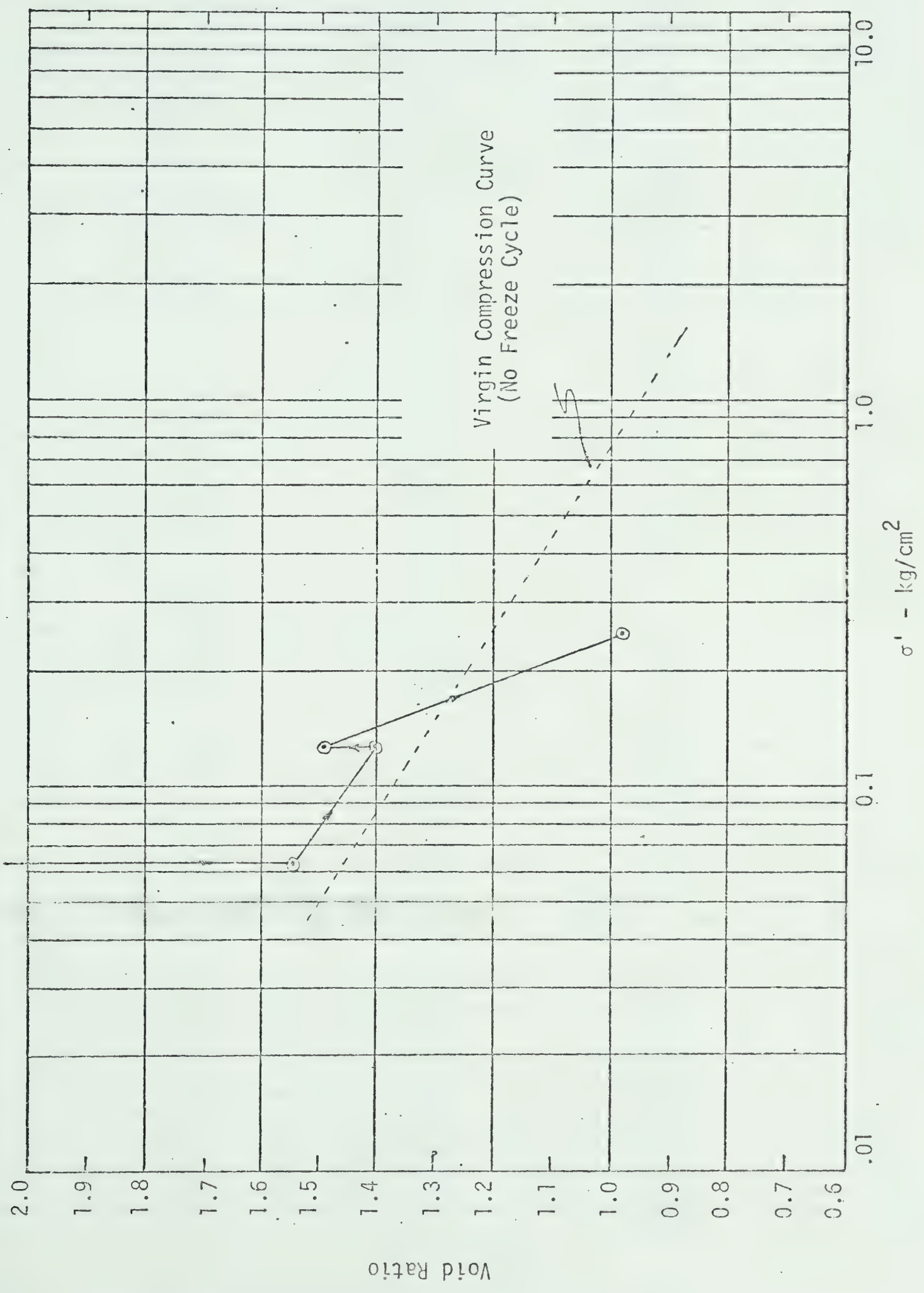


Fig. (D.1) Void Ratio versus effective stress Test Series 5-5

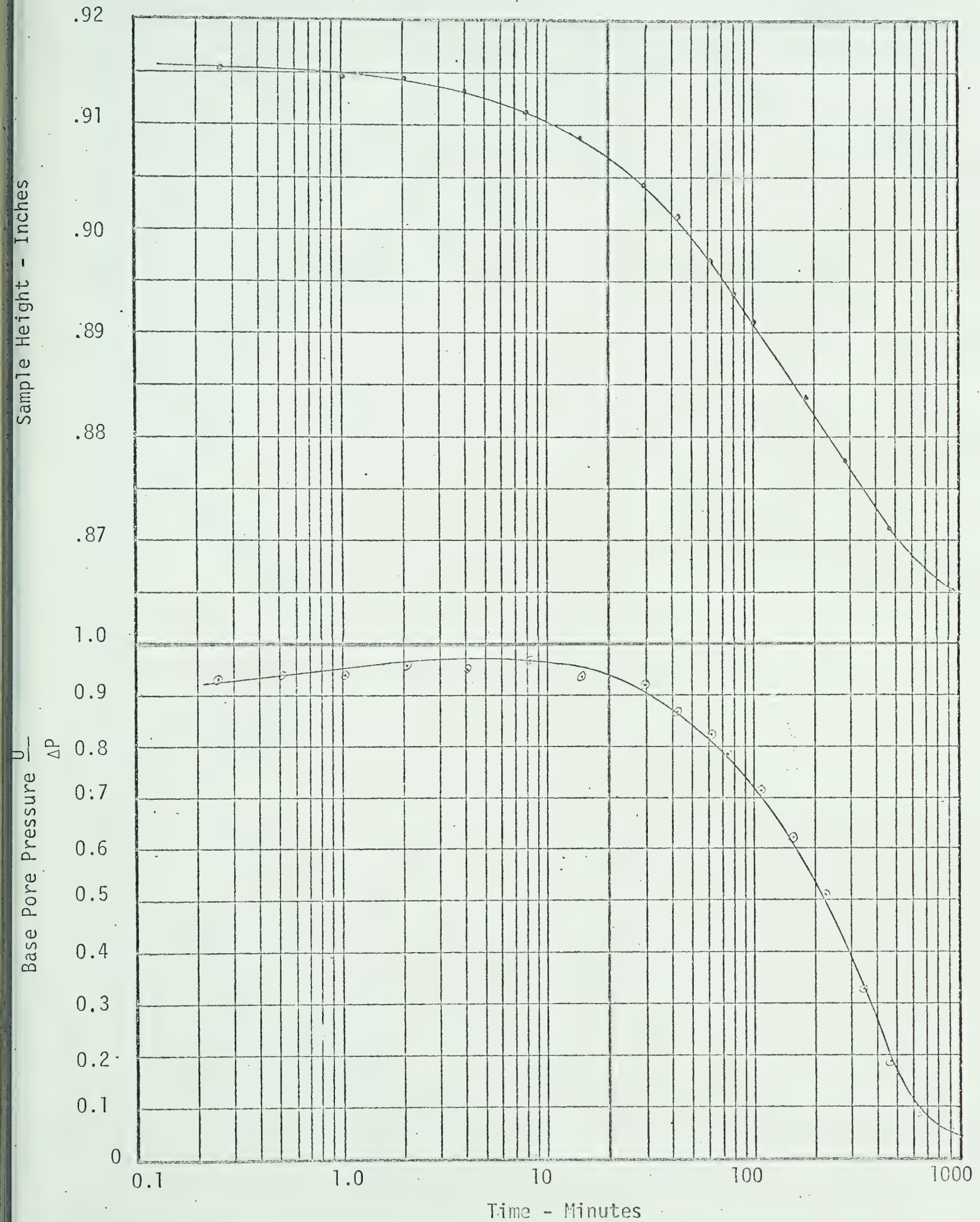
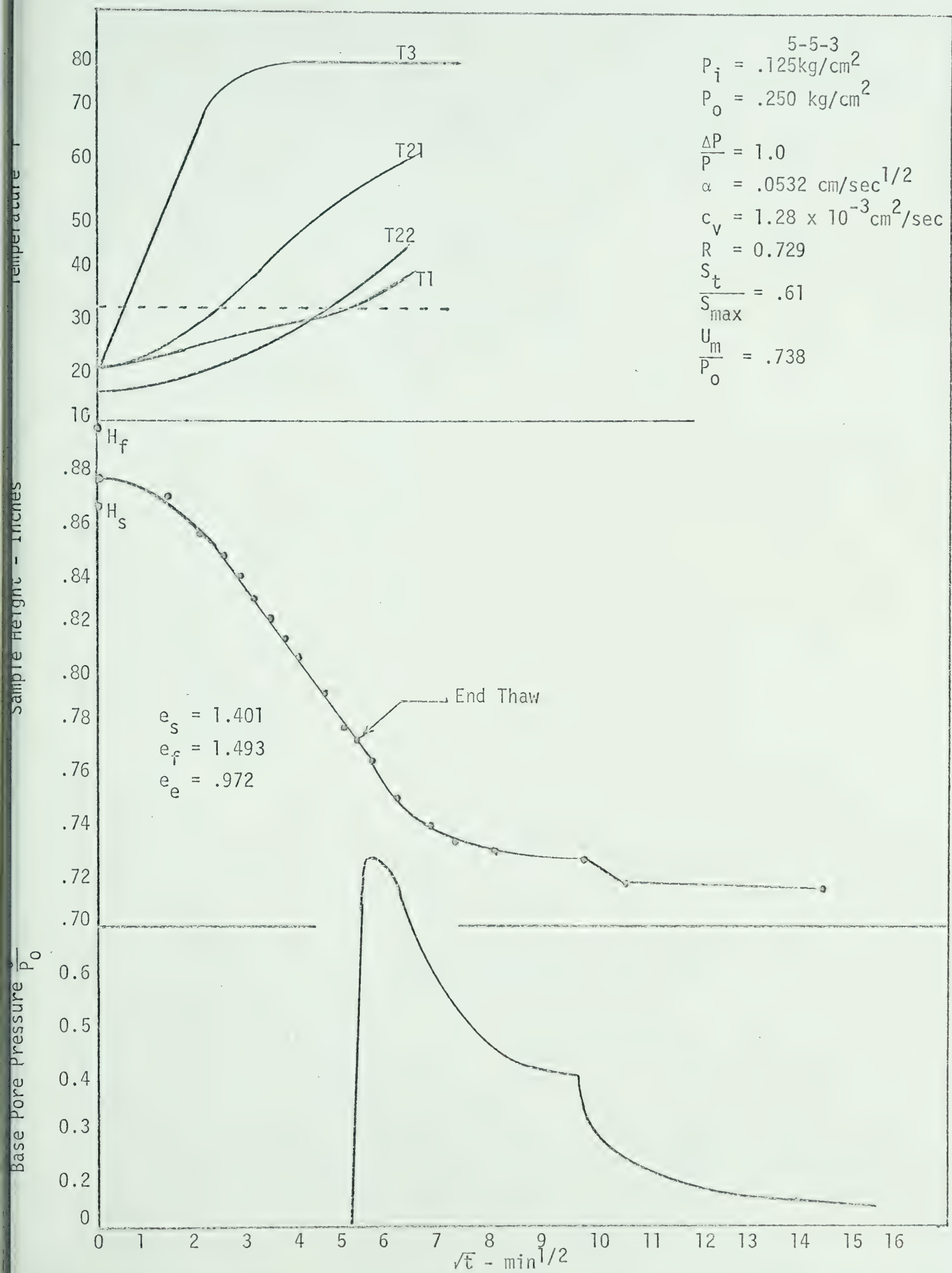


Fig. (D.2) Settlement and Pore Pressure Dissipation Curves Test 5-5-2



APPENDIX E

SUMMARY OF DATA

Test Series 5 - 6

APPENDIX E - TABLE 1

SUMMARY OF RESULTS
TEST SERIES 5-6

SOIL - ATHABASCA CLAY

Test	Phase	Press kg/cm ²	Dial in	H in	e	t ₅₀ min	k cm/sec x 10 ⁻³	c _v cm ² /sec x 10 ⁻³	m _v cm ² /gm x 10 ⁻³	$\sqrt{\text{Time}}$ (End Thaw) Min ^{1/2}	X (in)	$\frac{r}{\text{sec}}^{1/2}$	R = $\frac{a}{2\sqrt{c_v}}$	$\frac{S_t}{S_{\max}}$	$\frac{U_r}{P_0}$
5-6-1	Start	.062	.8076	1.4316	2.175	-	-	-	-	-	-	-	-	-	-
	End		.5025	1.1265	1.495										
5-6-2	Start	.125	.5025	1.1265	1.495	140	.535	.164	.326	-	-	-	-	-	-
	End		.4463	1.0703	1.390										
5-6-3	Start	.125	.4463	1.0703	1.390										
	Freeze	.250	.5031	1.1271	1.495										
	End Thaw		.3450	.9590	1.150										
	End		.2953	.9193	1.038	26	2.07	1.010	.210	7.48	1.0480	.0459	.722	$\frac{.1013}{.1510} = .67$	.735
5-6-4	Start	.250	.2953	.9193	1.038										
	Freeze	.250	.3357	.9597	1.123										
	End Thaw		.2675	.8915	.980	9.6	1.75	2.46	.071	6.48	.9256	.0468	.472	$\frac{.0278}{.0412} = .67$	.328
	End		.2541	.8781	.945										
5-6-5	Start	.250	.2541	.8781	.945										
	Freeze	.935	.2924	.9164	1.032										
	End Thaw		.2292	.8532	.890	12.3	0.70	1.71	.041	5.15	.8848	.0563	.681	$\frac{.0249}{.0623} = .40$	.515
	End		.1918	.8158	.822										
5-6-6	Start	.935	.1918	.8158	.822										
	Freeze	2.0	.2240	.8480	.870										
	End Thaw		.1750	.7990	.772	16.5	0.22	1.09	.020	4.69	.8235	.0576	.872	$\frac{.0168}{.0470} = .36$	.764
	End		.1448	.7688	.704										
5-6-7	Start	2.0	.1448	.7688	.704										
	Freeze	2.0	.1645	.7835	.748										
	End Thaw		.1342	.7582	.682	4.3	.04	4.00	.001	11.4	.7734	.0222	.176	$\frac{.0106}{.0123} = .86$	.132
	End		.1325	.7565	.677										

APPENDIX E - TABLE 1

(CONTINUED)

Test	Phase	Press kg/cm ²	Dial in	H in	e	t ₅₀ min	k cm/sec x 10 ⁻⁷	c _v cm ² /sec x 10 ⁻³	m _{v2} /gm x 10 ⁻³	$\sqrt{\text{Time}}$ (End Thaw) Min ^{1/2}	X (in)	α cm/sec ^{1/2}	$R = \frac{\alpha}{2\sqrt{c_v}}$	$\frac{S_t}{S_{max}}$	$\frac{u}{p_0}$
5-6-8	Start	2.0	.1325	.7565	.677										
	Freeze	2.0	.1669	.7909	.754										
	End Thaw		.1370	.7610	.688	1.7	0.50	10.10	.005	4.12	.7760	.0618	.307	-	.175
	End		.1281	.7521	.670										
5-6-9	Start	2.0	.1281	.7521	.670										
	Freeze	2.0	.1552	.7792	.729										
	End Thaw		.1267	.7507	.662	2.4	0.14	7.0	.002	4.47	.7650	.0551	.335	$\frac{.0014}{.0054} = .26$	.154
	End		.1227	.7467	.655										
5-6-10	Start	2.0	.1227	.7467	.655										
	Freeze	2.0	.1591	.7831	.736										
	End Thaw		.1280	.7520	.665	10.5	0.06	1.61	.004	6.64	.7676	.0375	.472	-	.344
	End		.1214	.7454	.654										
5-6-11	Start	2.0	.1214	.7454	.654										
	Freeze	2.0	.1425	.7665	.700										
	End Thaw		.1207	.7447	.652	2.0	.024	8.25	.003	4.47	.7556	.0554	.305	$\frac{.0007}{.0045} = .16$	.017
	End		.1169	.7409	.642										

END OF TEST DATA

$\omega = 24.25\%$
 $W_s = 96.66 \text{ gm}$
Final ht = .7409 in
 $H_0 = .4510 \text{ in}$
 $S = 100\%$

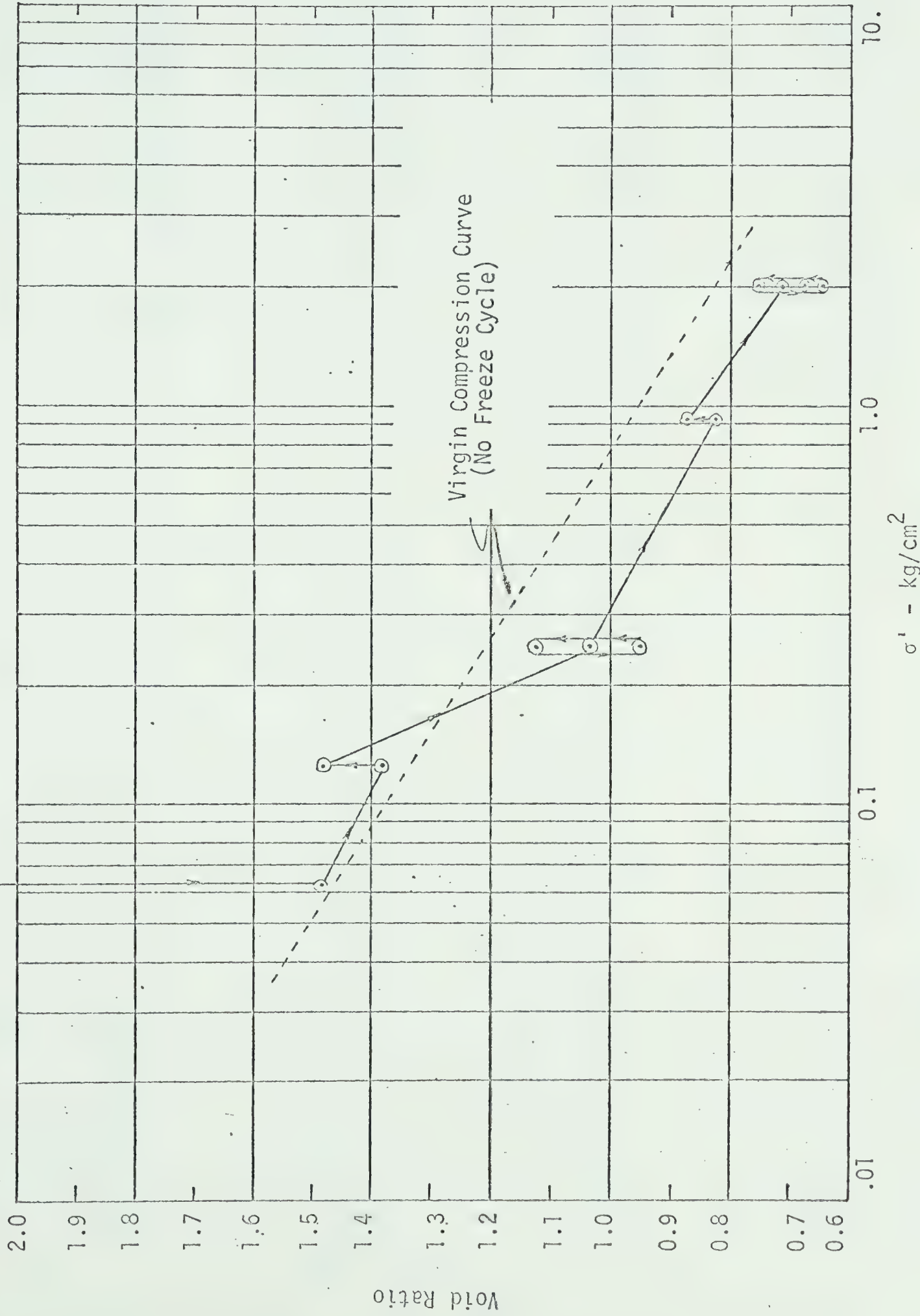


Fig. (E.1) Void Ratio versus effective Stress - Test Series 5-6

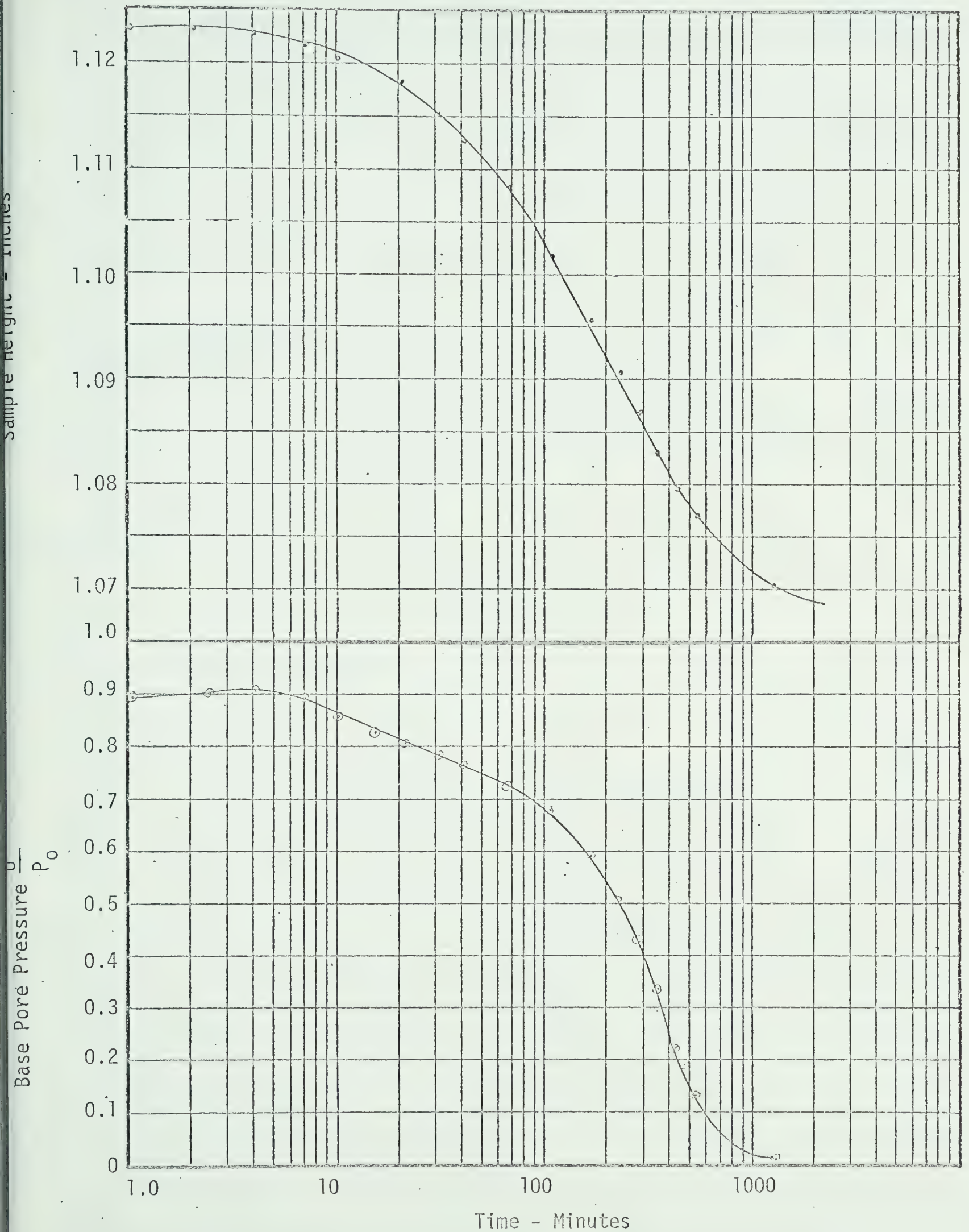


Fig. (E.2) Settlement and Pore Pressure Dissipation Curves - Test 5-6-2

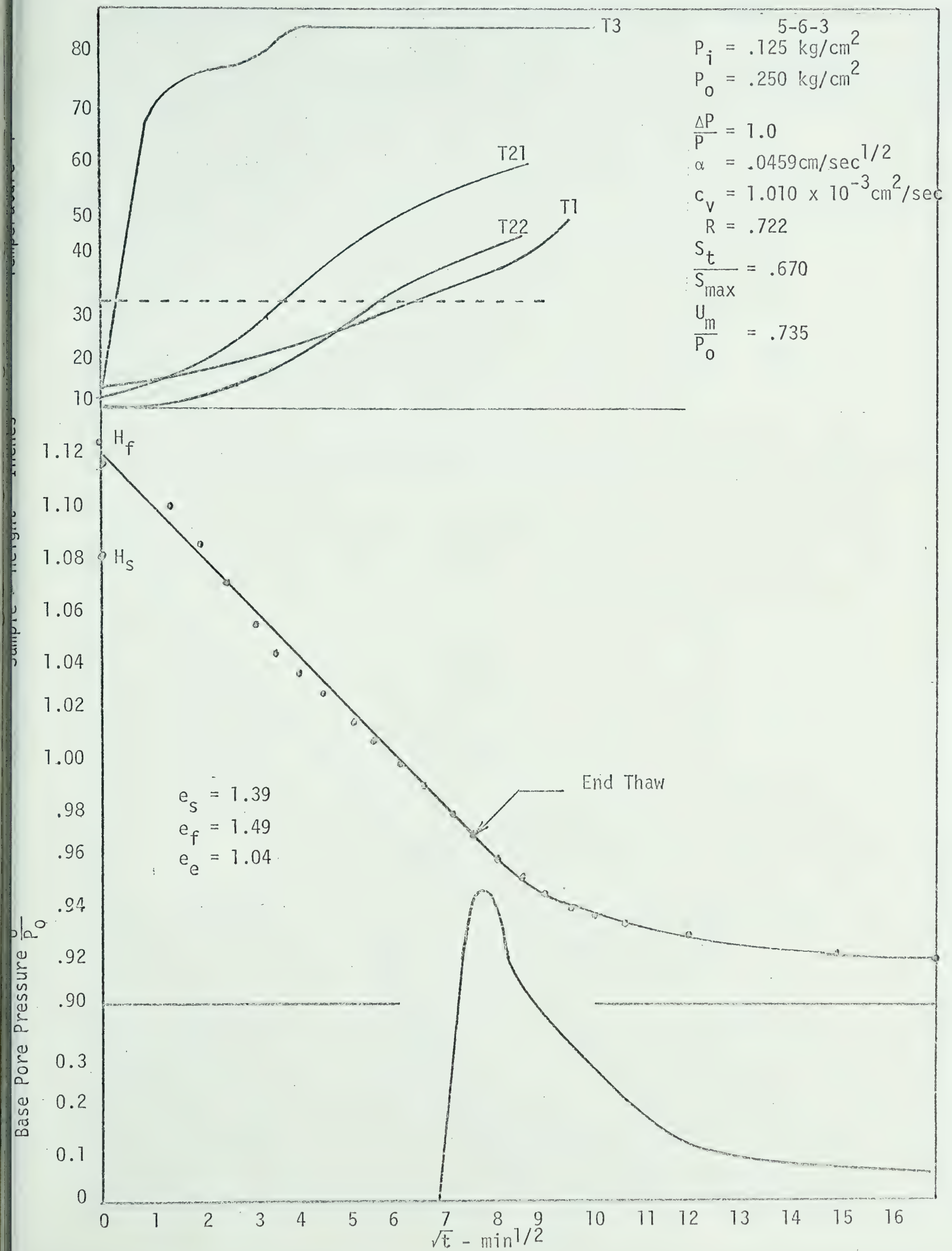
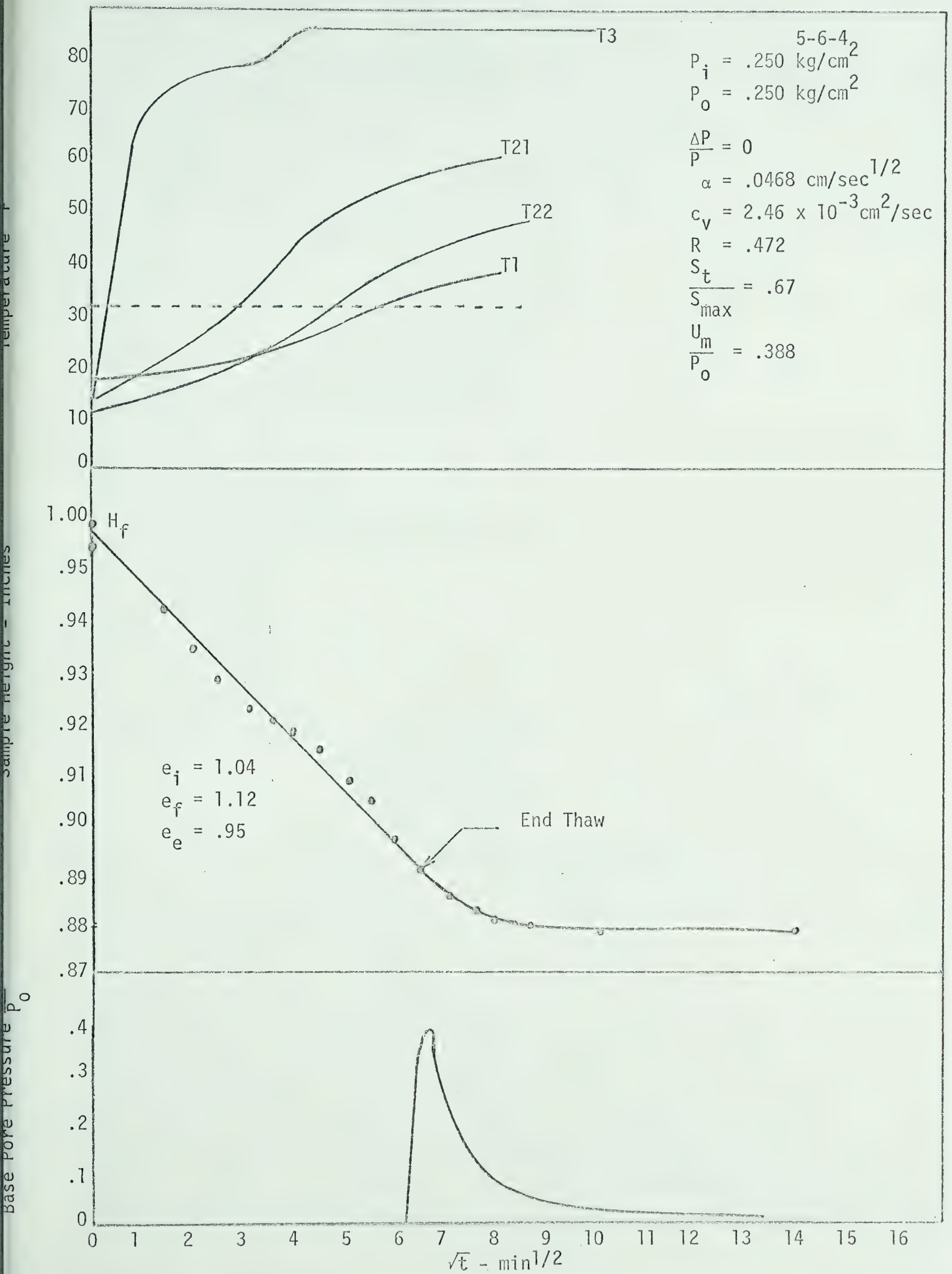
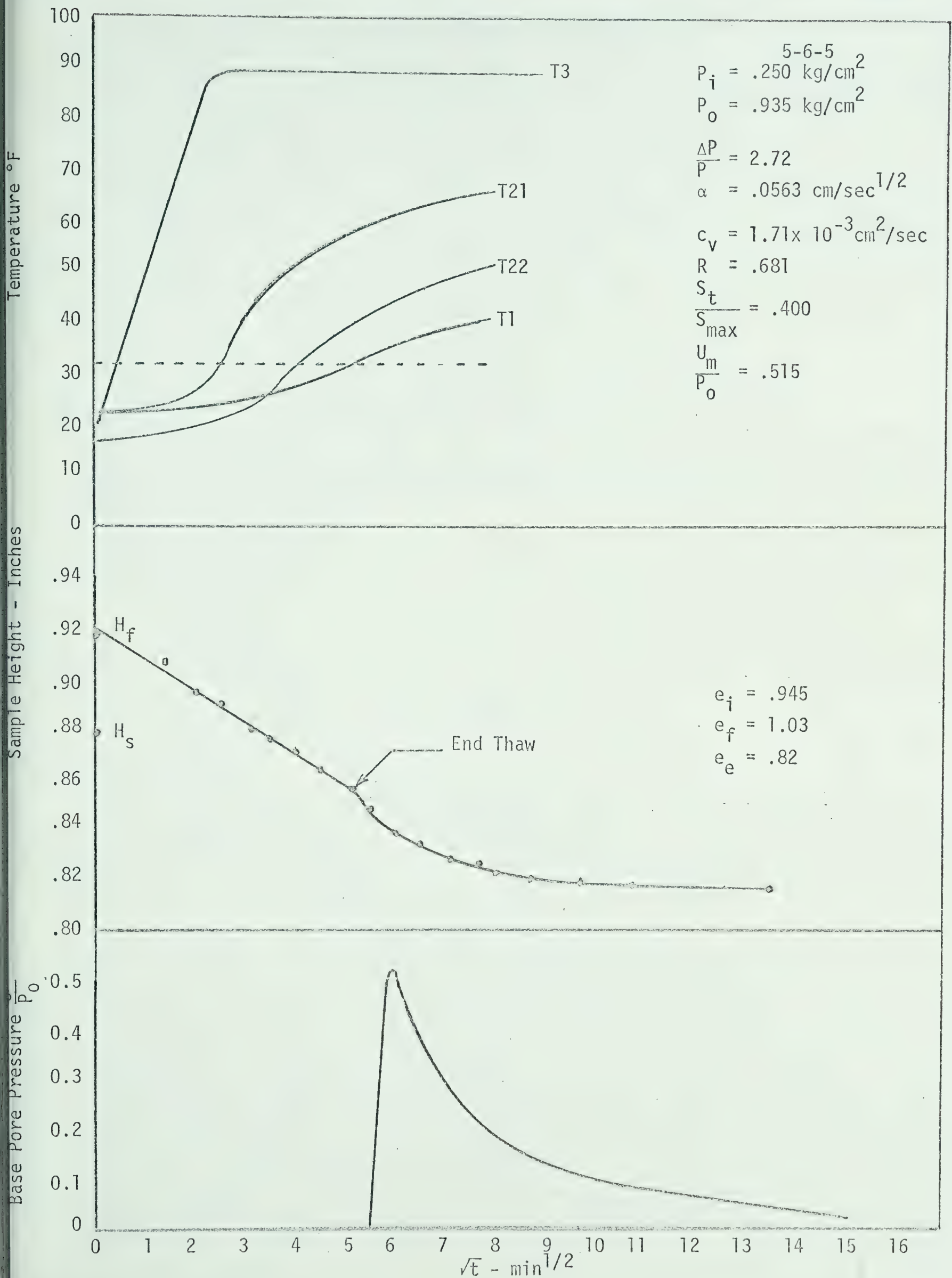
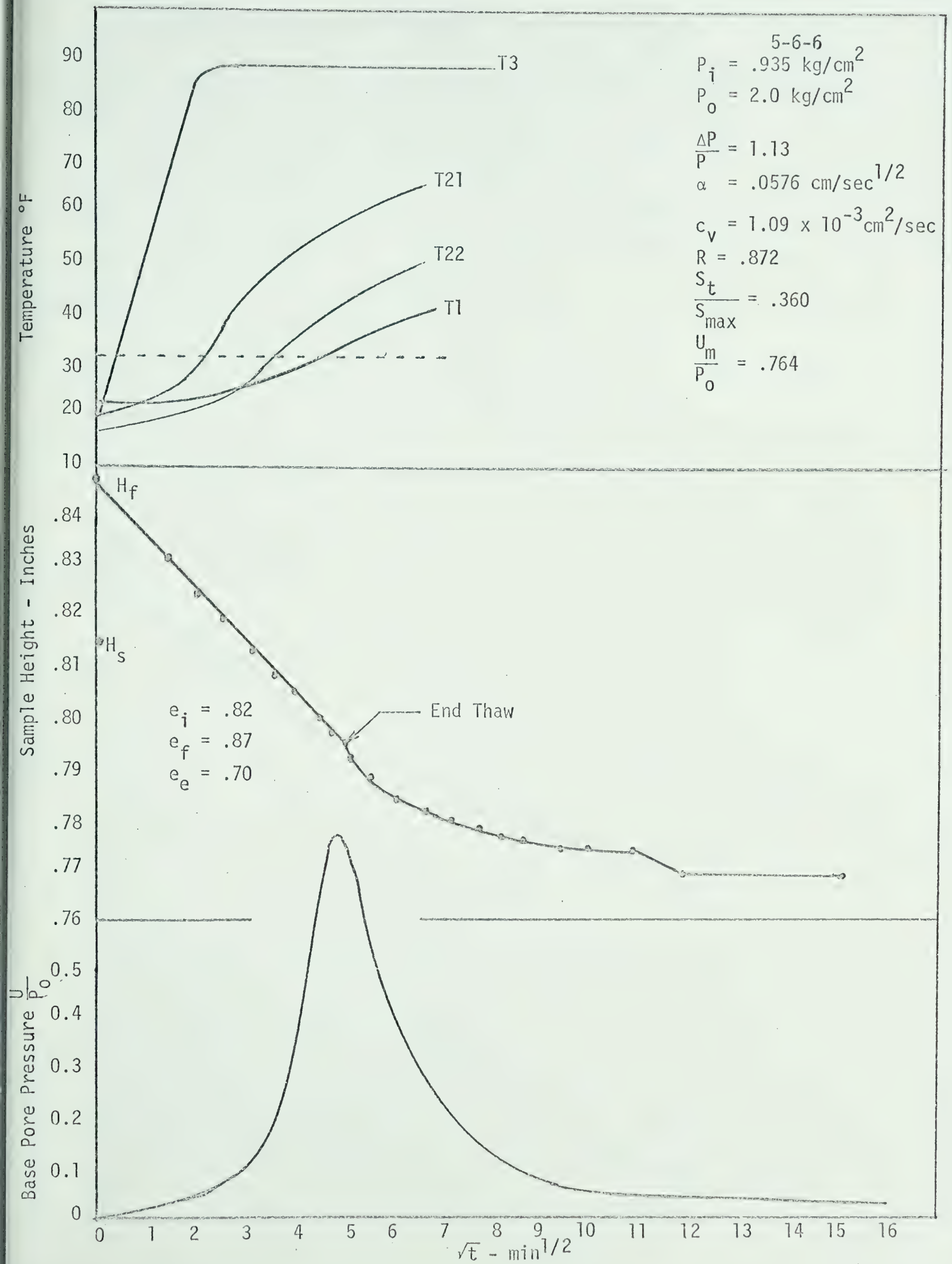


Fig. (E.3)







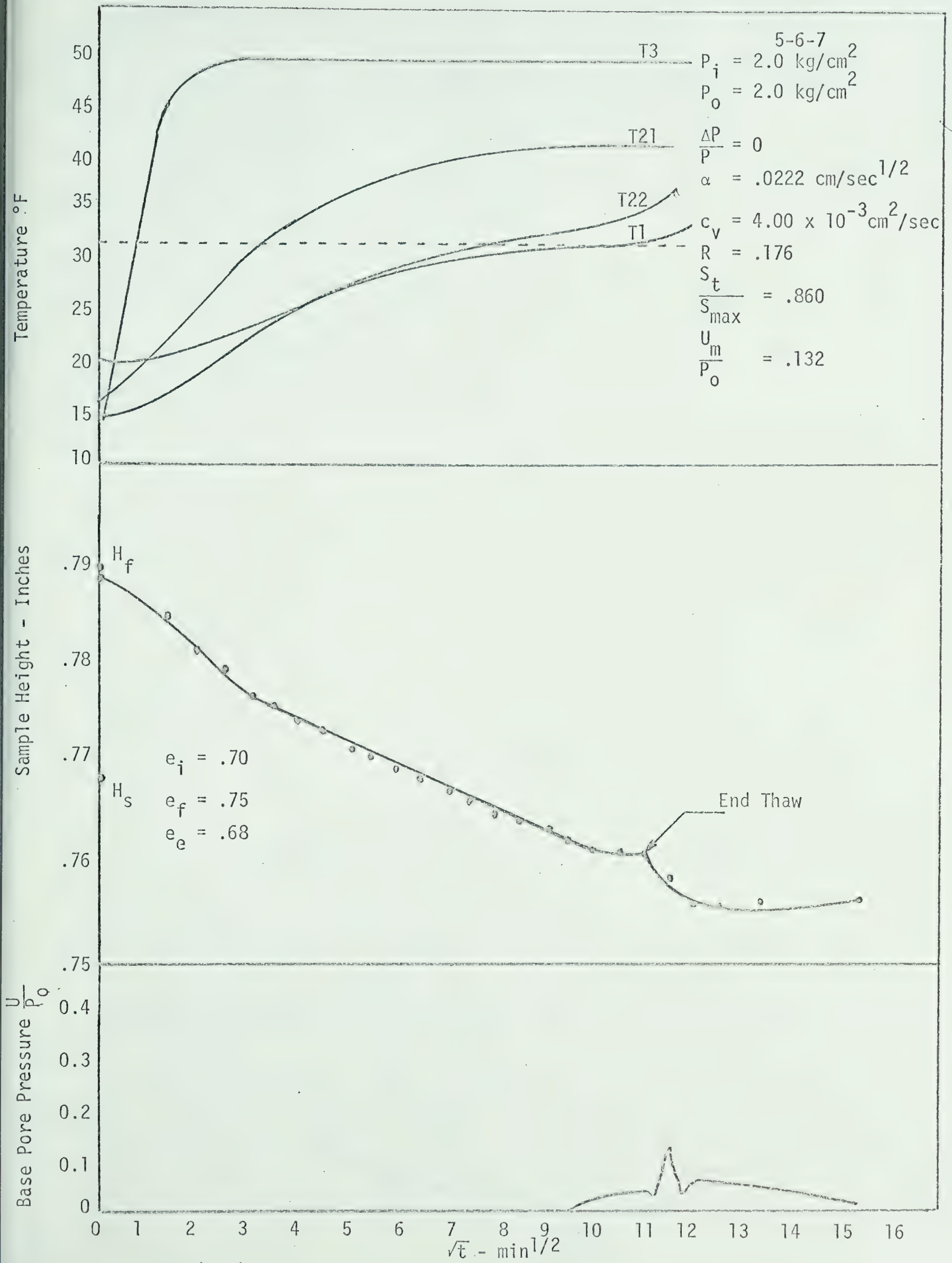
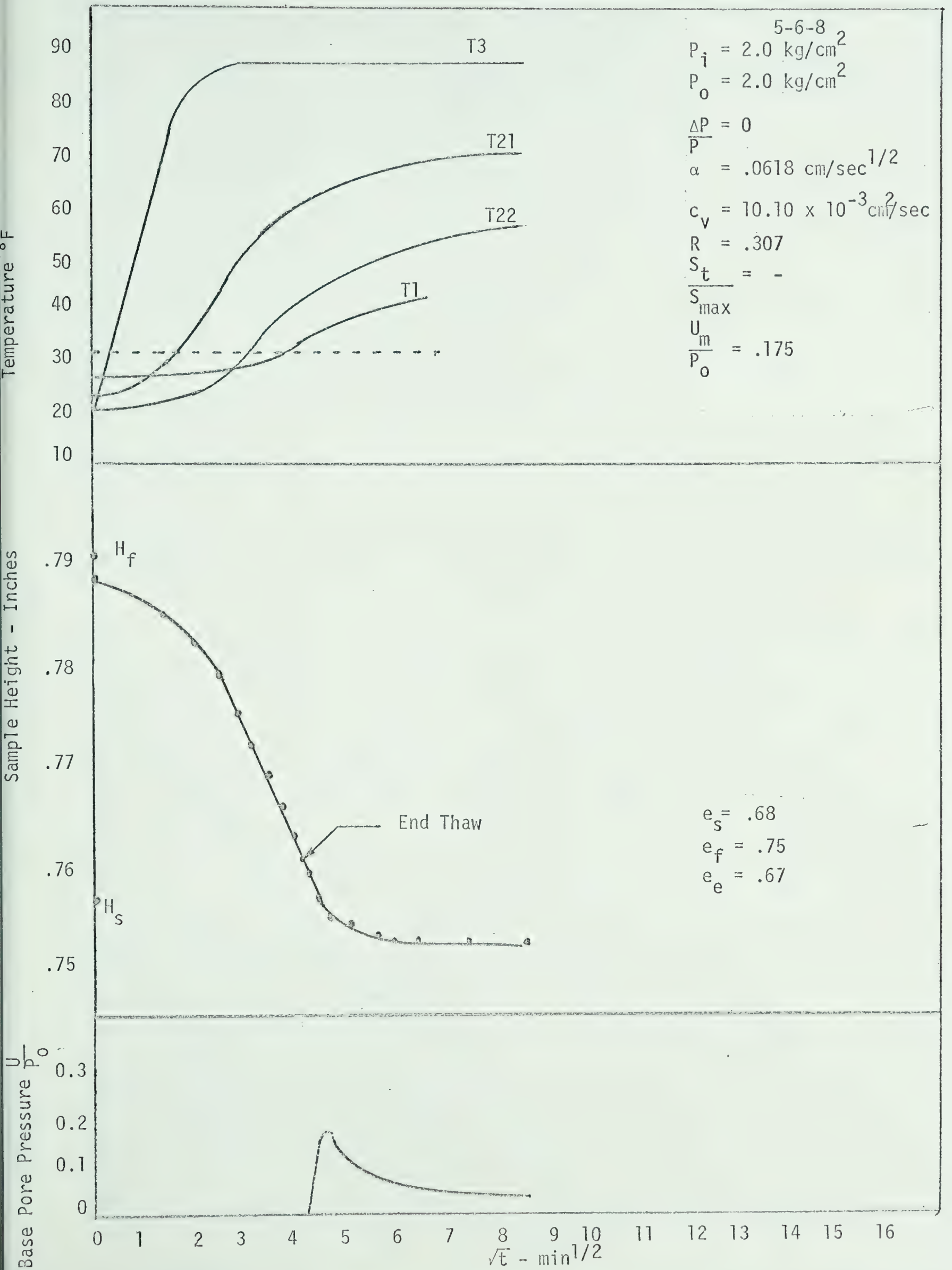
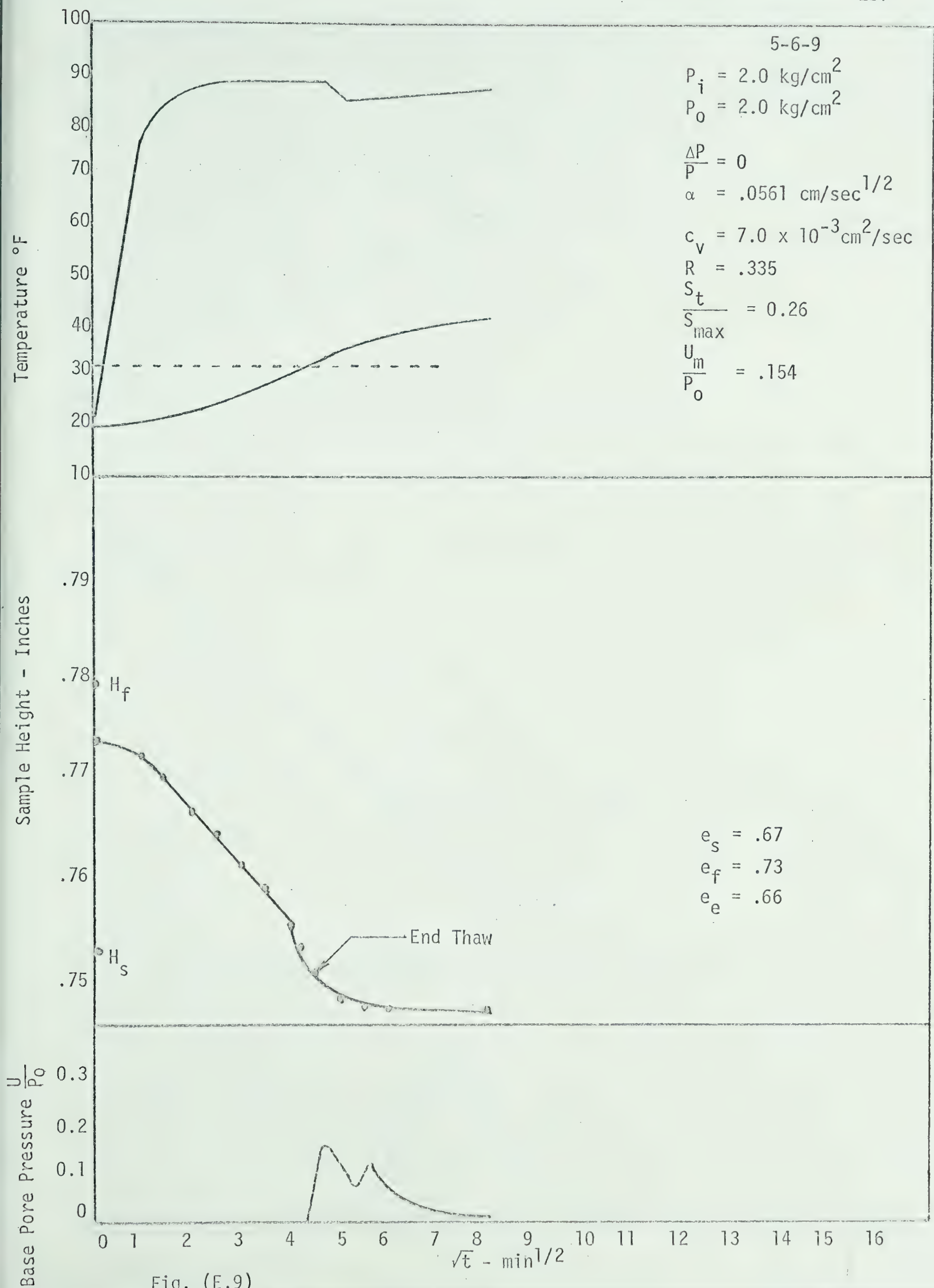
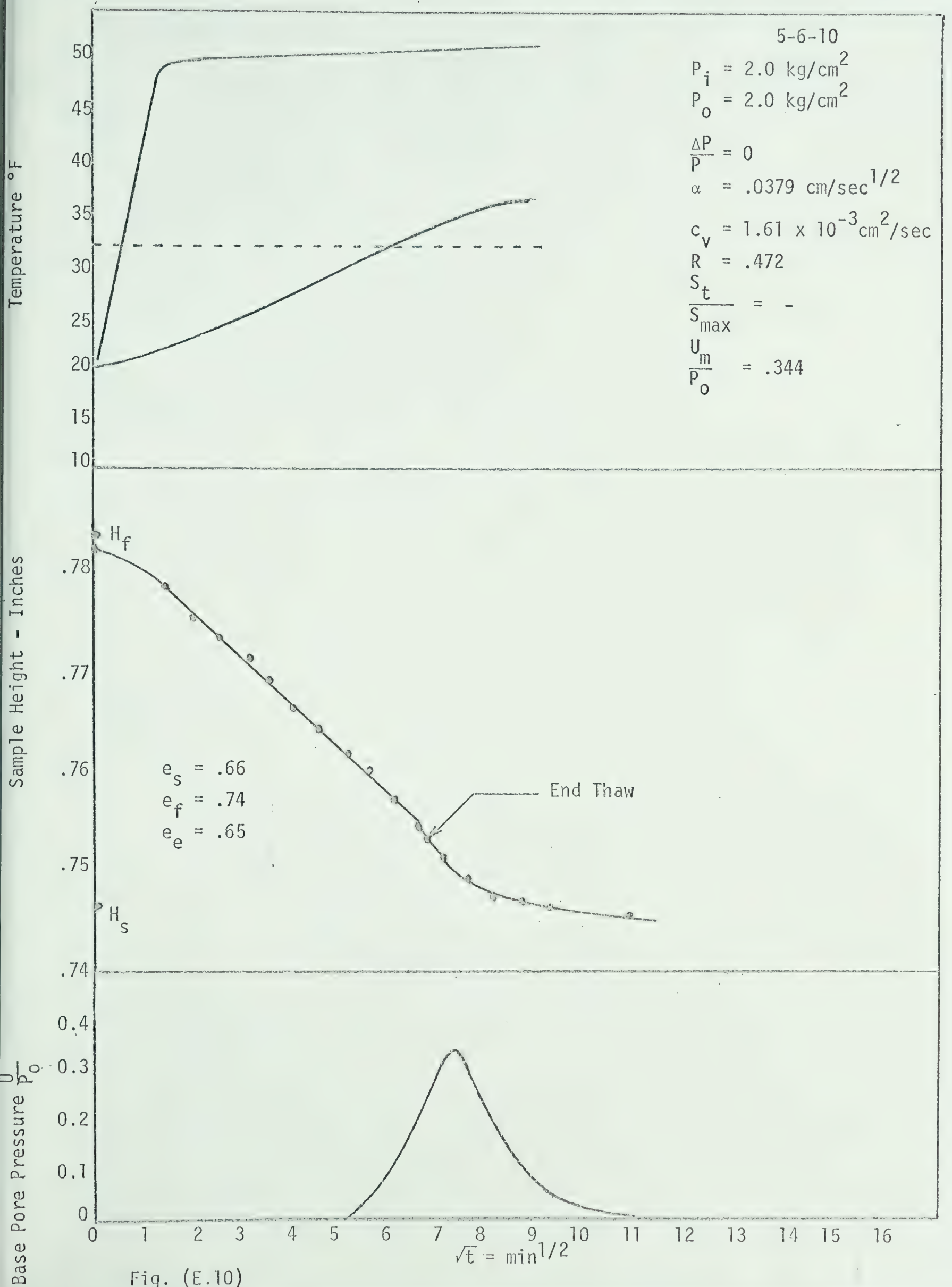


Fig. (E.7)







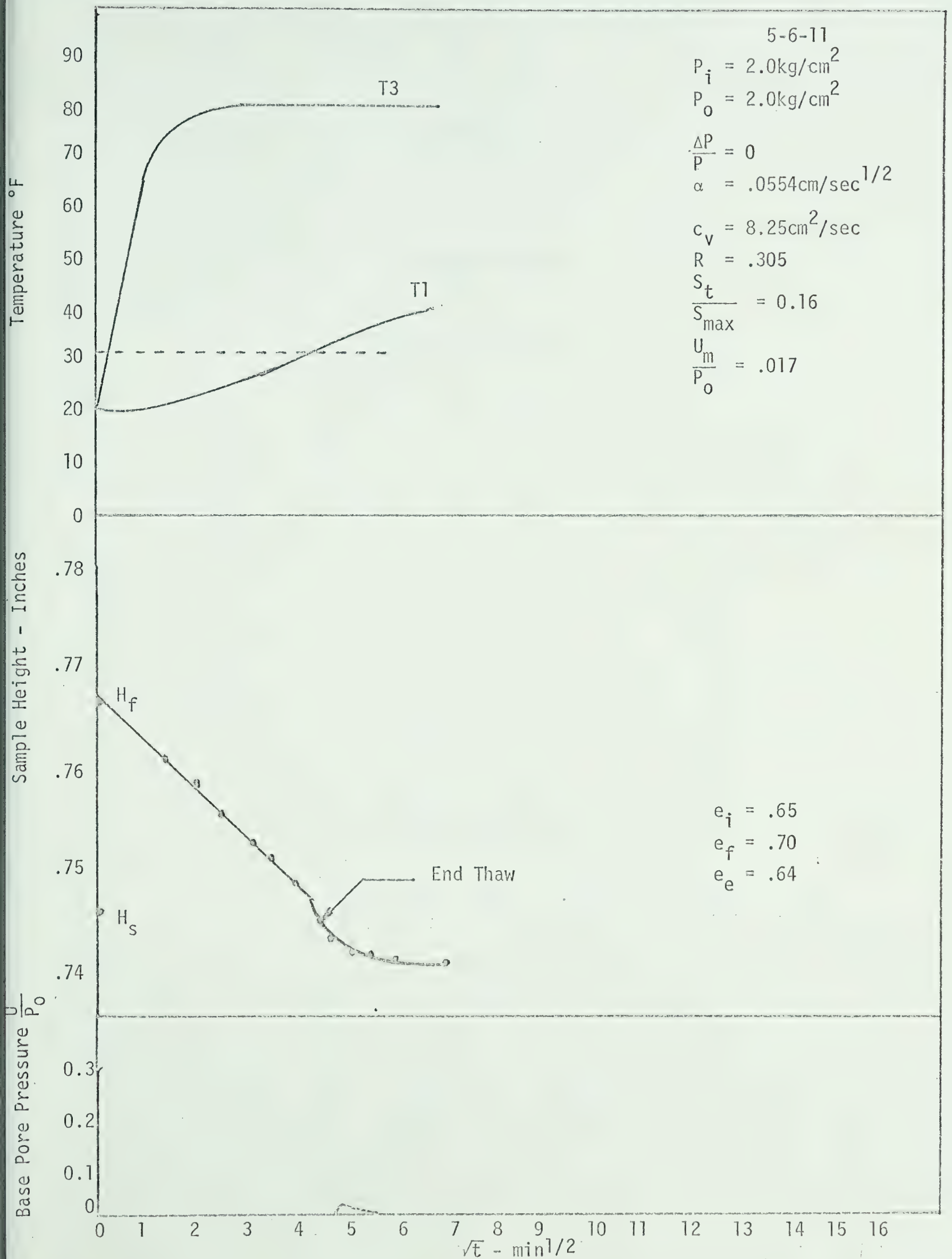


Fig. (E.11)

APPENDIX F

SUMMARY OF DATA

Test Series 5 - 7

APPENDIX F - TABLE 1

SUMMARY OF RESULTS
TEST SERIES 5-7

SOIL - ATHABASCA CLAY

Test	Phase	Press kg/cm ²	Dial in	H in	e	t ₅₀ min	k cm/sec x 10 ⁻⁷	c _v cm ² /sec x 10 ⁻³	m _{v2} cm/gm x 10 ⁻³	$\sqrt{\text{Time}}$ (End Thaw) min ^{1/2}	X (in)	α cm/sec ^{1/2}	$R = \frac{\alpha}{2\sqrt{c_v}}$	$\frac{S_t}{S_{max}}$	$\frac{U_t}{P_0}$
5-7-1	Start	0	.8700	1.4112	2.744										
	Freeze	0.125	.9663	1.5080	3.001										
	End Thaw		.4620	1.0032	1.662	5	98.3	4.75	2.07	6.93	1.2556	.0594	.431	.639	.992
	End		.2320	.7732	1.050										
5-7-2	Start	0.125	.2160	.7592	1.014										
	Freeze	0.250	.2517	.7929	1.104										
	End Thaw		.1816	.7228	.918	45	4.30	3.39	.127	3.32	.7579	.0749	.643	.617	.620
	End		.1590	.7002	.858										
5-7-3	Start	0.25	.1555	.6967	.848										
	Freeze	0.50	.1881	.7293	.935	Not Defined	-	-	-	-	-	-	-	-	.202
	End Thaw		Not defined	-	-										
	End		.1176	.6588	.748										
5-7-4	Start	0.688	.1120	.6532	.733	24	.398	.369	.108	-	-	-	-	-	-
	End		.0989	.6401	.698										
5-7-5	Start	1.685	.0986	.6398	.698	11	.185	.769	.024	-	-	-	-	-	-
	End		.0836	.6248	.658										
5-7-6	Start	2.389	.0792	.6204	.646	24.5	.237	.315	.0753	-	-	-	-	-	-
	End		.0472	.5884	.561										
5-7-7	Start	0	.0457	.5869	.557	Not Defined	-	-	-	-	-	-	-	-	-
	End	0	.1076	.6488	.721										

END OF TEST DATA

L = 28.05%

Final ht = .6488 in

S = 103.0%

W_s = 80.29 gmH_o = .3769 in

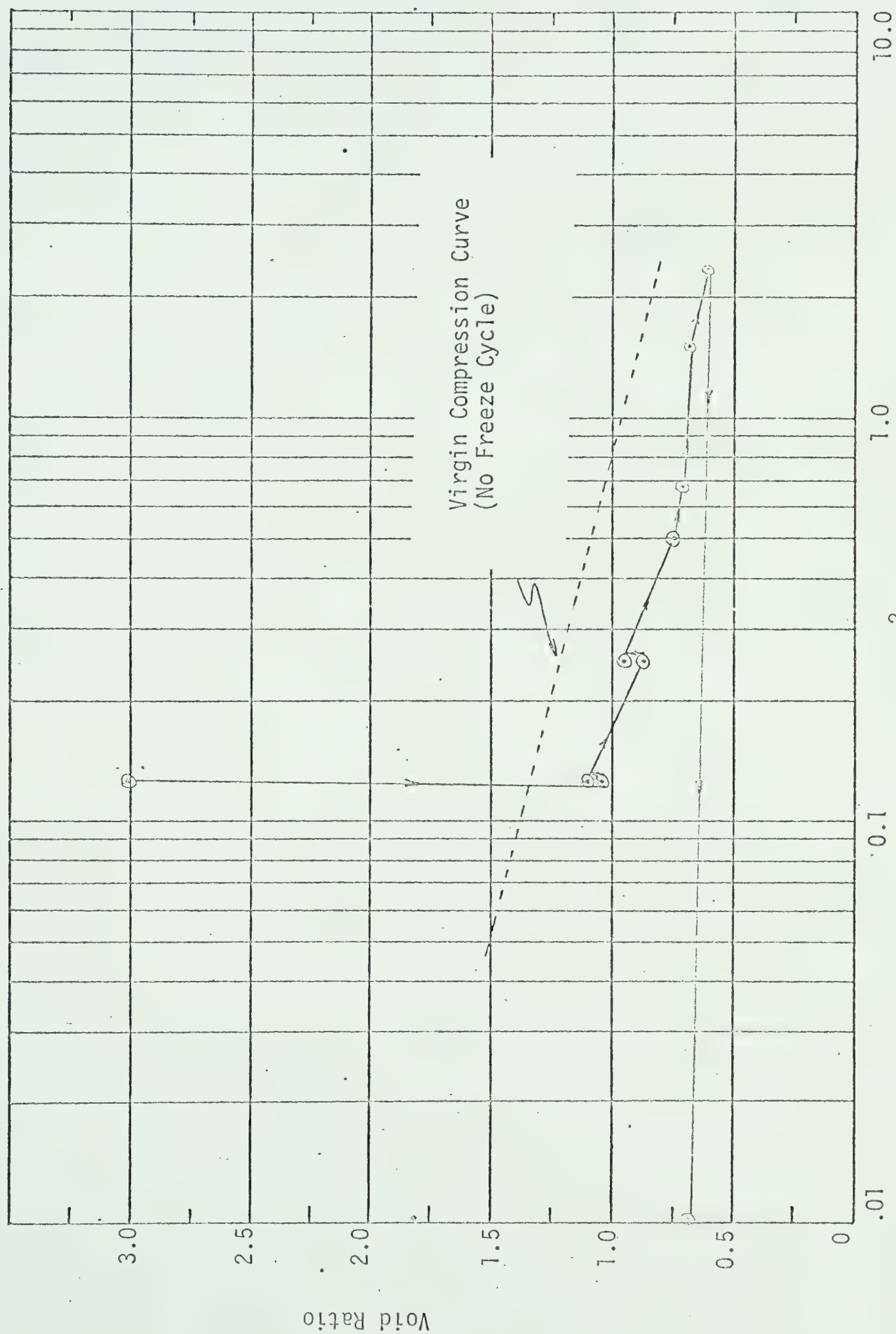


Fig. (F.1) Void Ratio versus Effective Stress Test Series 5-7 (Athabasca Clay)

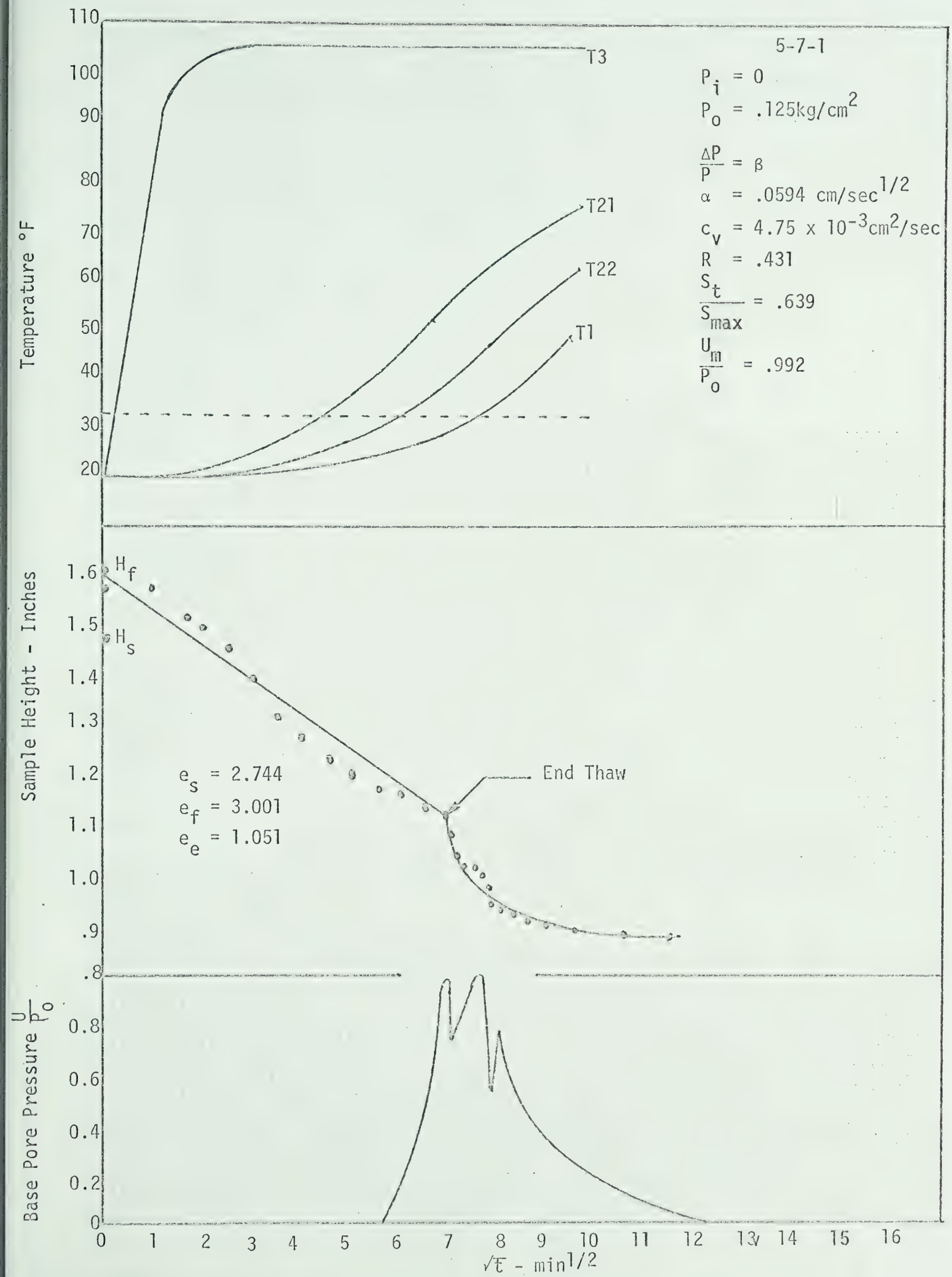
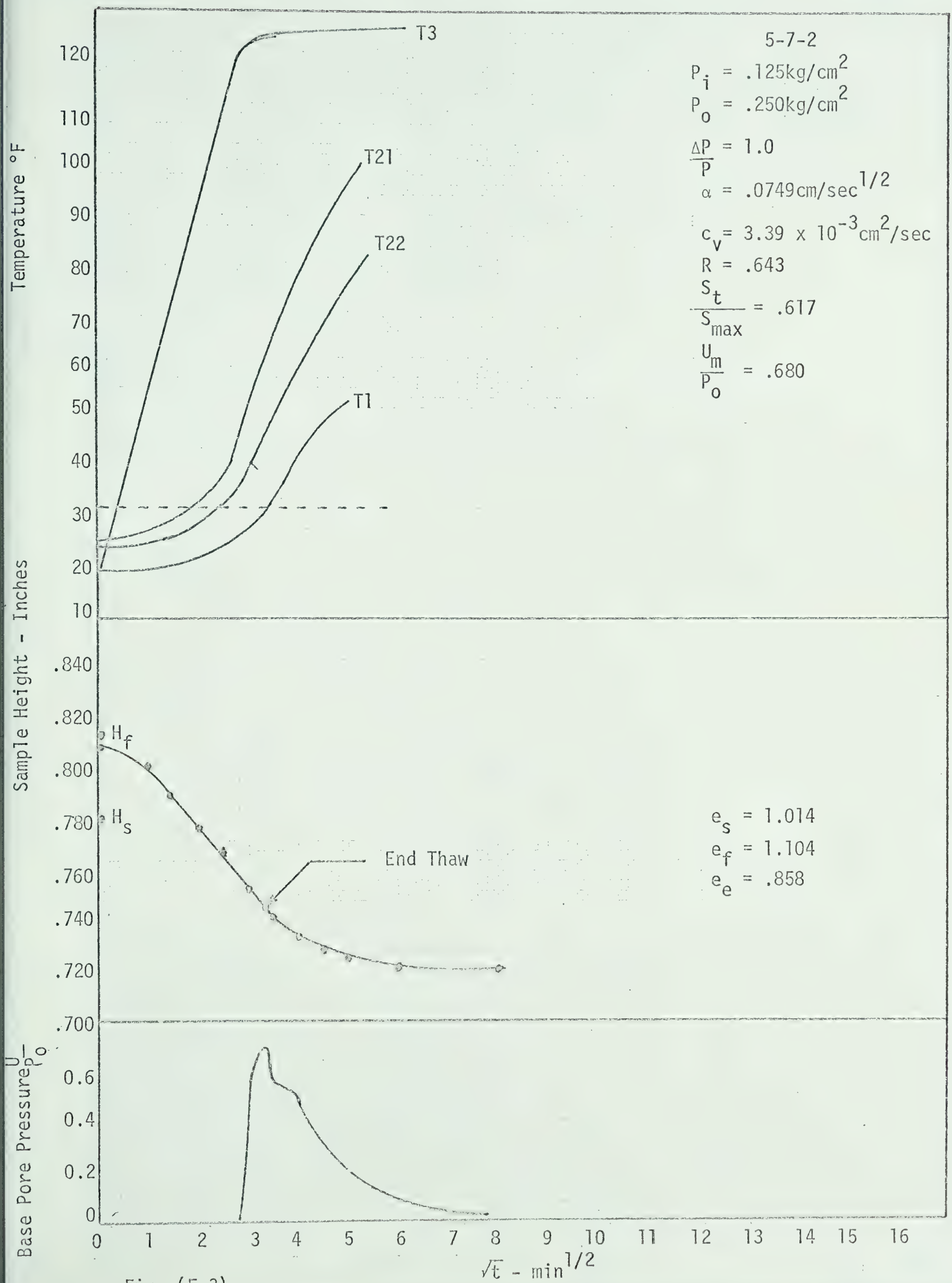
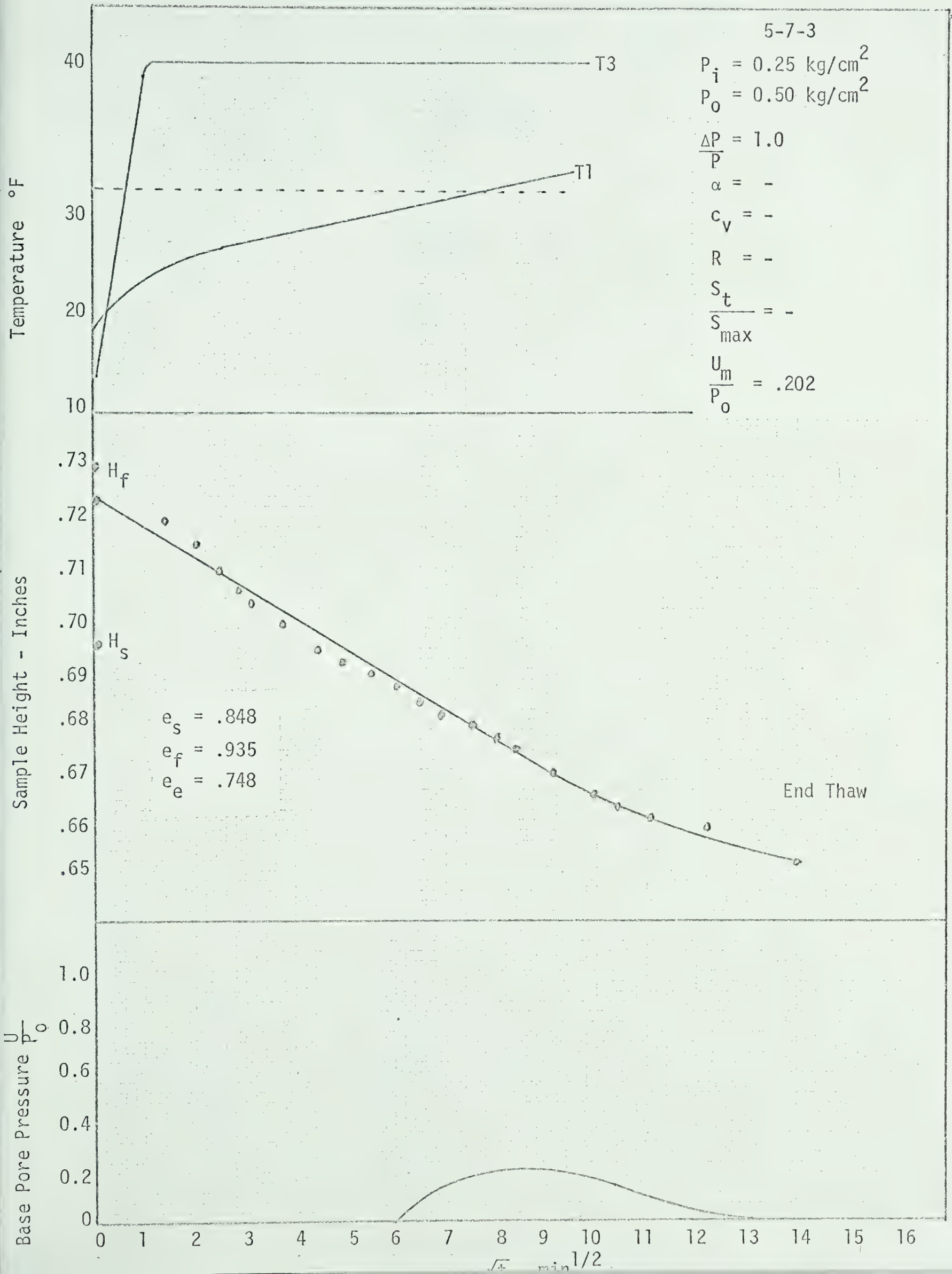


Fig. (F.2)





APPENDIX G

SUMMARY OF DATA

Test Series 5 - 8

APPENDIX G - TABLE 1

SUMMARY OF RESULTS
TEST SERIES 5-4

SOIL - LAKE EDMONTON CLAY

Test	Phase	Press kg/cm ²	Dial in	H in	e	t ₅₀ min	k cm/sec x 10 ⁻⁷	c _v cm ² /sec x 10 ⁻³	m _v cm ² /sec x 10 ⁻³	$\sqrt{\frac{t}{t_0}}$ (End Thaw) in ^{1/2}	X (in)	α cm/sec ^{1/2}	$R = \frac{\alpha}{2\sqrt{c_v}}$	$\frac{s_t}{s_{max}}$	$\frac{U_m}{P_0}$
5-8-1	Start	0	.7444	1.3218	3.807										
	Freeze	.063	.8032	1.3806	4.020										
	End Thaw		.2940	.8714	2.169			1.79	1.732	7.74	.1260	.0477	.564	.833	.380
	End		.2040	.7814	1.841	11.5	3.10								
5-8-2	Start	.063	.1945	.7719	1.807										
	Freeze	.442	.2307	.8081	1.938										
	End Thaw		.0972	.6746	1.453			2.69	.136	3.87	.7414	.0628	.605	.711	.578
	End		.0576	.6350	1.31	4.8	3.66								

END OF TEST DATA

$\omega = 47.5\%$
 $W_s = -$
Final Ht = 0.6350 in
 $H_0 = 0.275$ in
S = 100% (assumed)

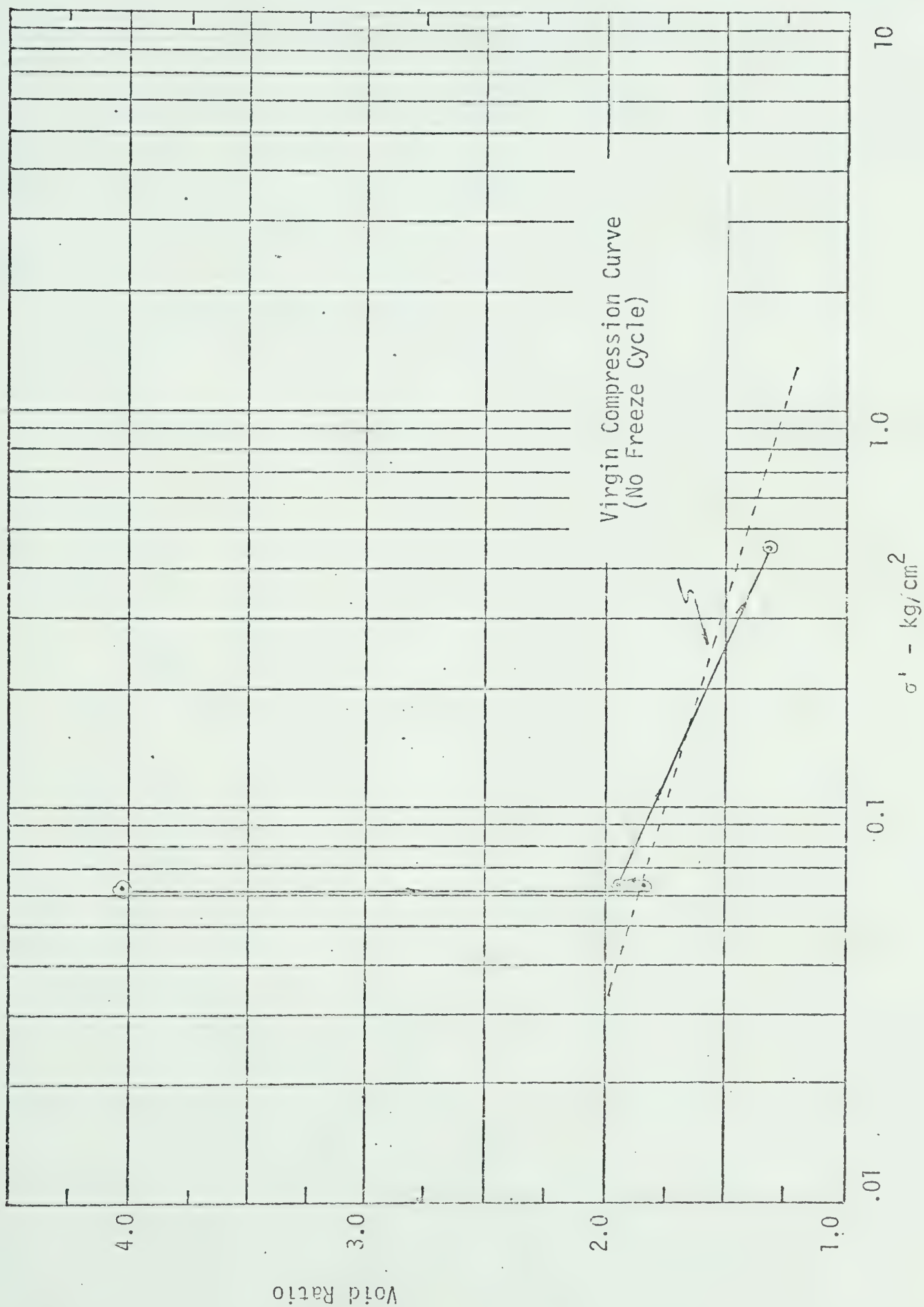


Fig. (G.1) Void ratio versus Effective Stress - Test Series (Lake Edmonton Clay)

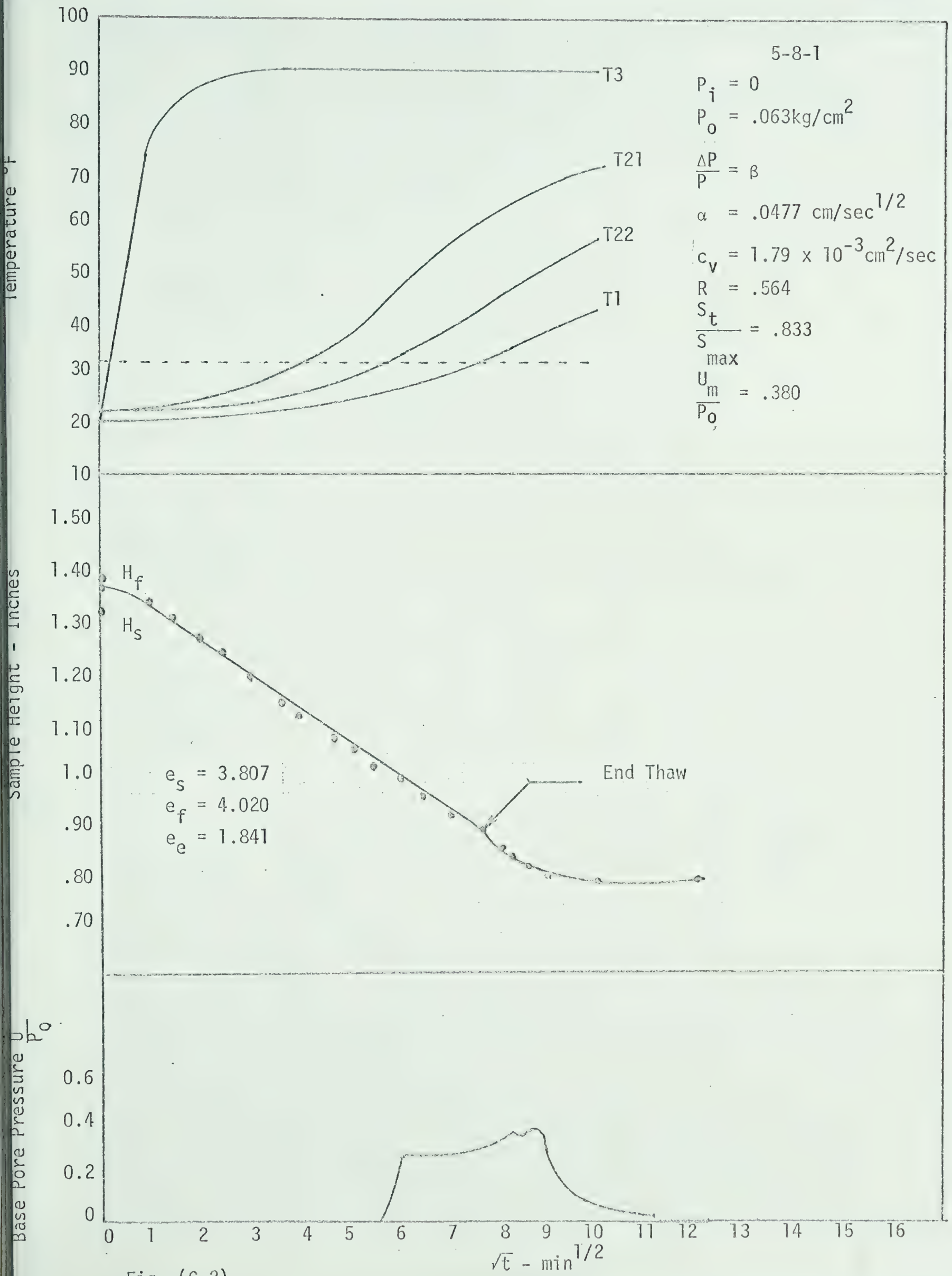


Fig. (G.2)

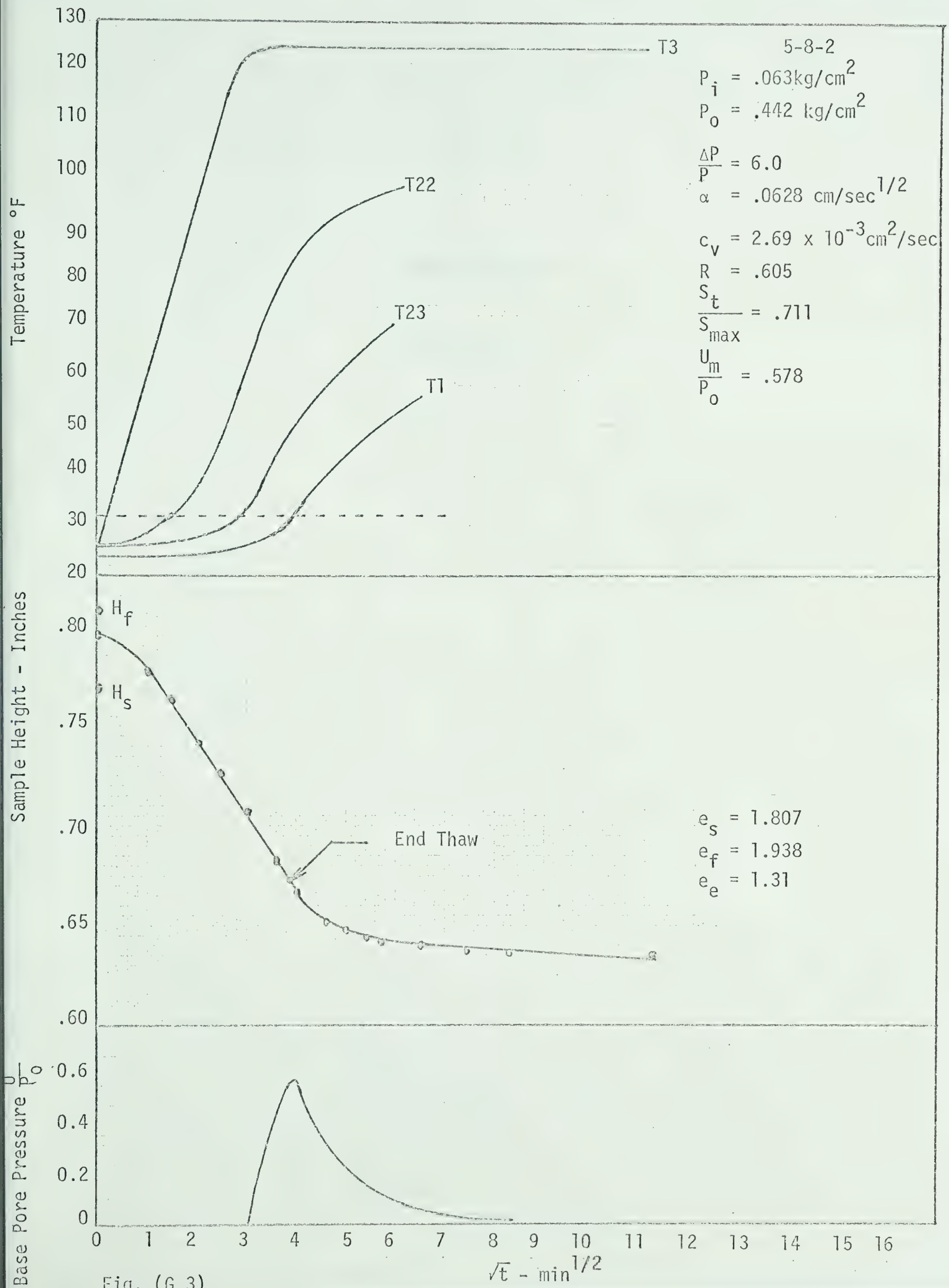


Fig. (G.3)

APPENDIX H

SUMMARY OF DATA

Test Series 5 - 9

APPENDIX H - TABLE 1

SUMMARY OF RESULTS
TEST SERIES 5-9

SOIL - BENTONITE

Test	Phase	Press kg/cm ²	Dial in	H in	e	t ₅₀ min	k cm/sec x 10 ⁻⁹	c _v cm ² /sec x 10 ⁻⁶	m _v cm ² /sec x 10 ⁻³	$\sqrt{\frac{\text{Time}}{\text{End}}}$ min ^{1/2}	X (in)	α cm/sec ^{1/2}	$R = \frac{\alpha}{2\sqrt{c_v}}$	$\frac{S_t}{S_{max}}$	$\frac{U_m}{P_0}$
5-9-1	Start	.063	1.422	1.4185	17.45										
	Freeze	.313	1.522	1.5185	18.75										
	End Thaw		1.104	1.1005	13.31	2000	13.49	8.08	1.67	9.48	1.3095	.0453	7.97	.410	.967
	End		.647	0.6435	7.37										
5-9-2	Start	.313	.631	0.6275	7.16										
	Freeze	.690	.669	0.6655	7.65										
	End Thaw		.602	0.5985	6.78	1200	3.29	4.43	.74	7.48	0.6320	.0277	6.58	.120	.914
	End		.3885	0.385	4.01										

END OF TEST DATA

$$\omega = 140.5\%$$

$$W_s = 17.00 \text{ gm}$$

$$\text{Final ht} = 0.385 \text{ in}$$

$$H_0 = 0.0769$$

$$S = 96.5\%$$

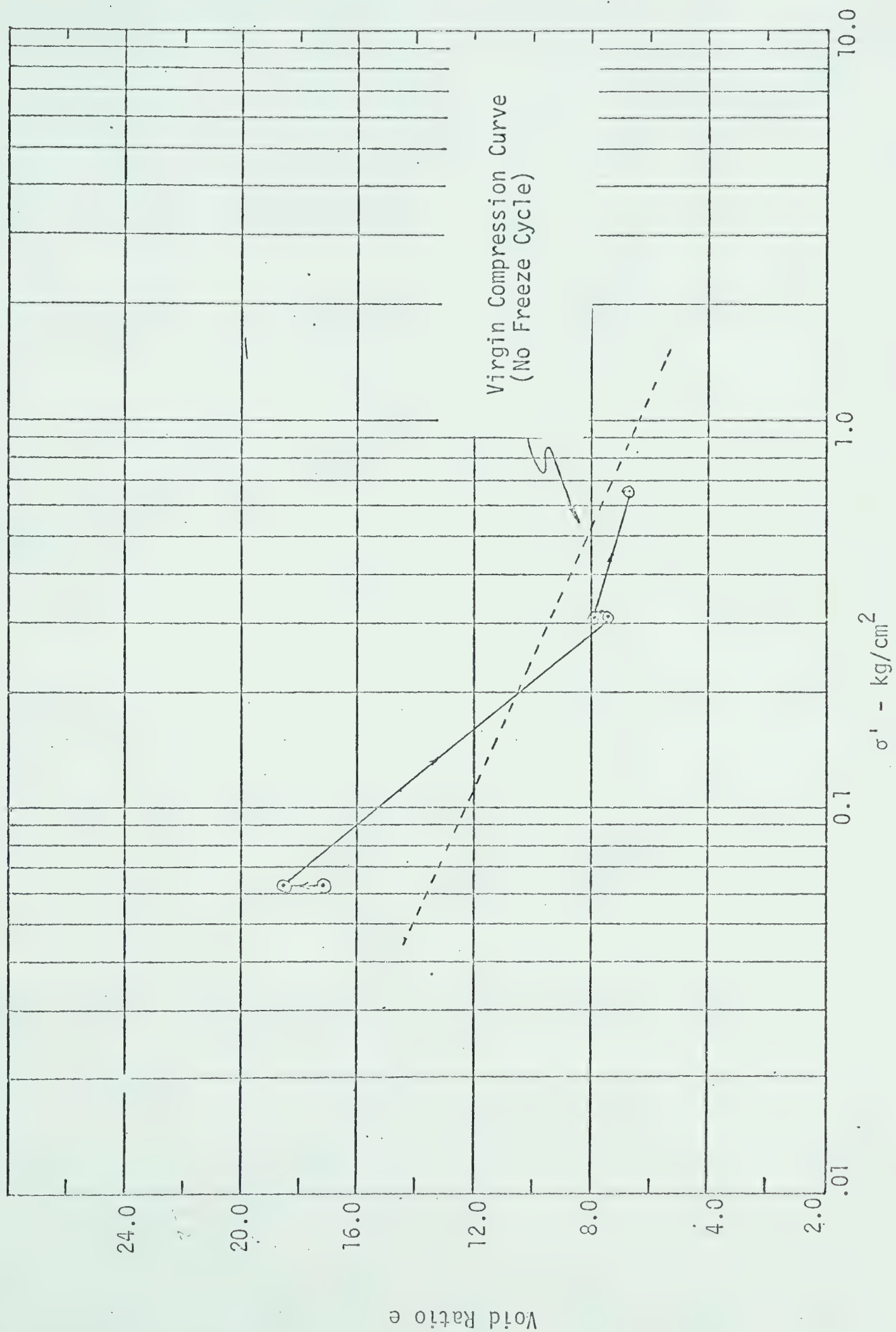
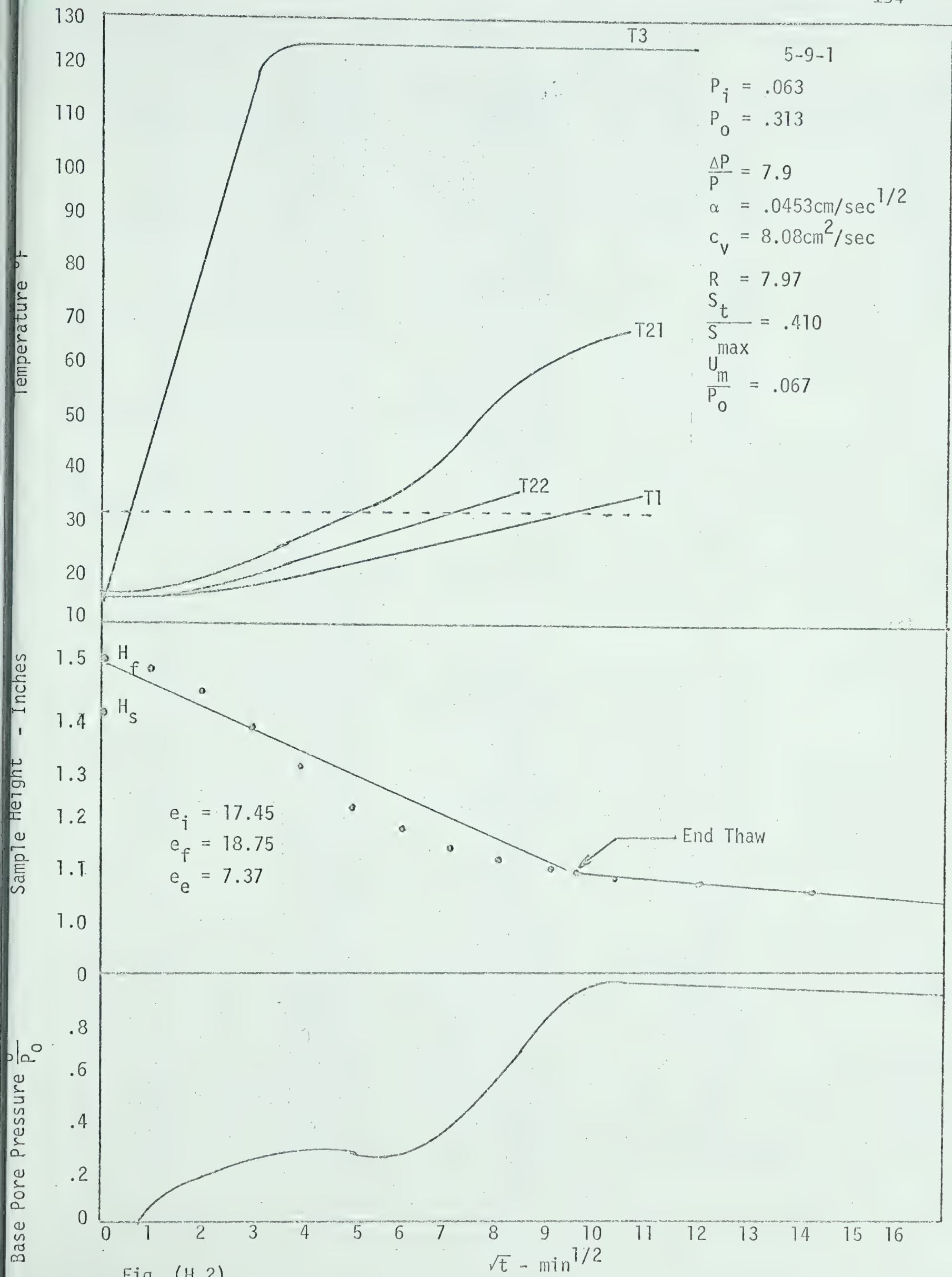


Fig. (H.1) Void Ratio versus Effective Stress - Test Series 5-9



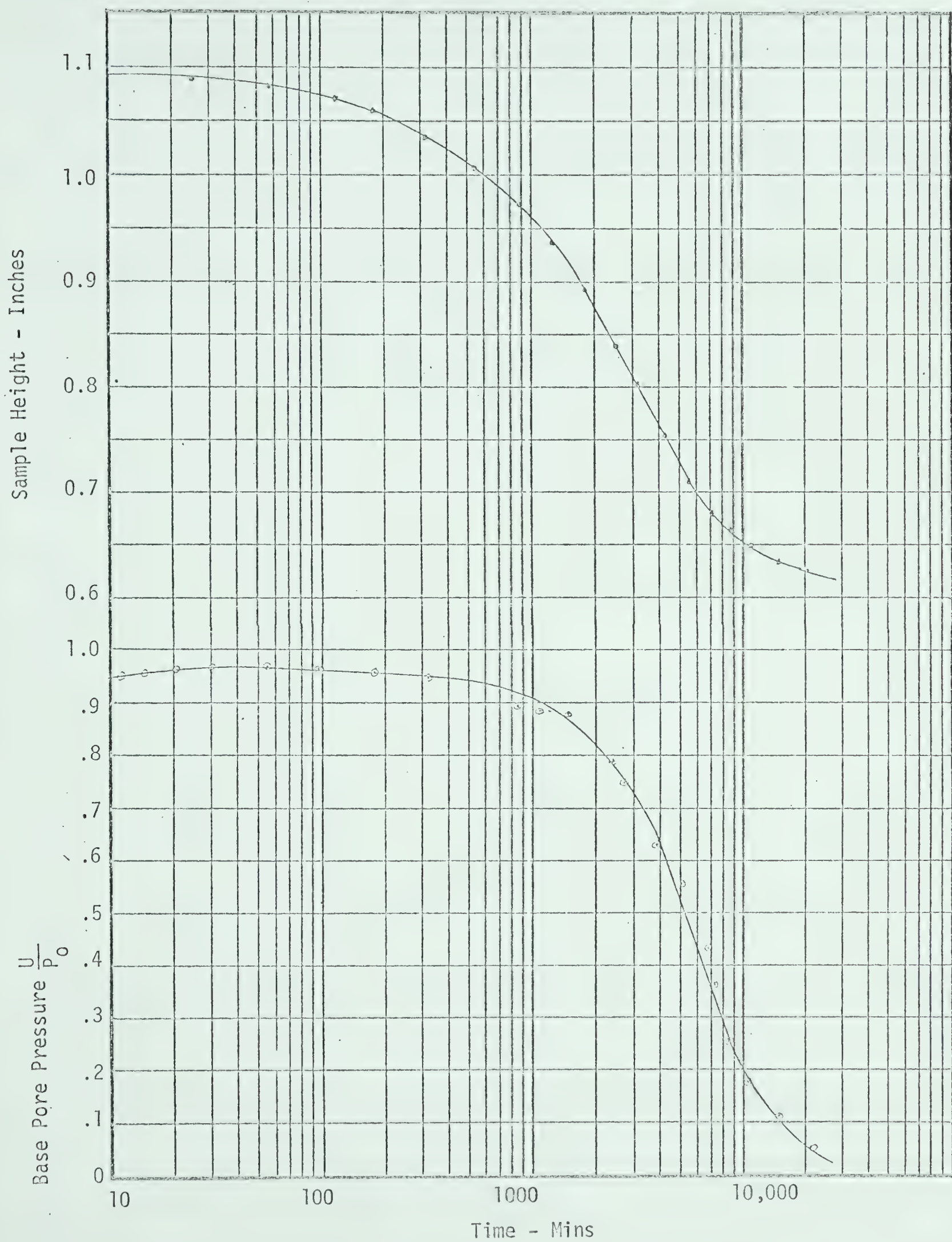
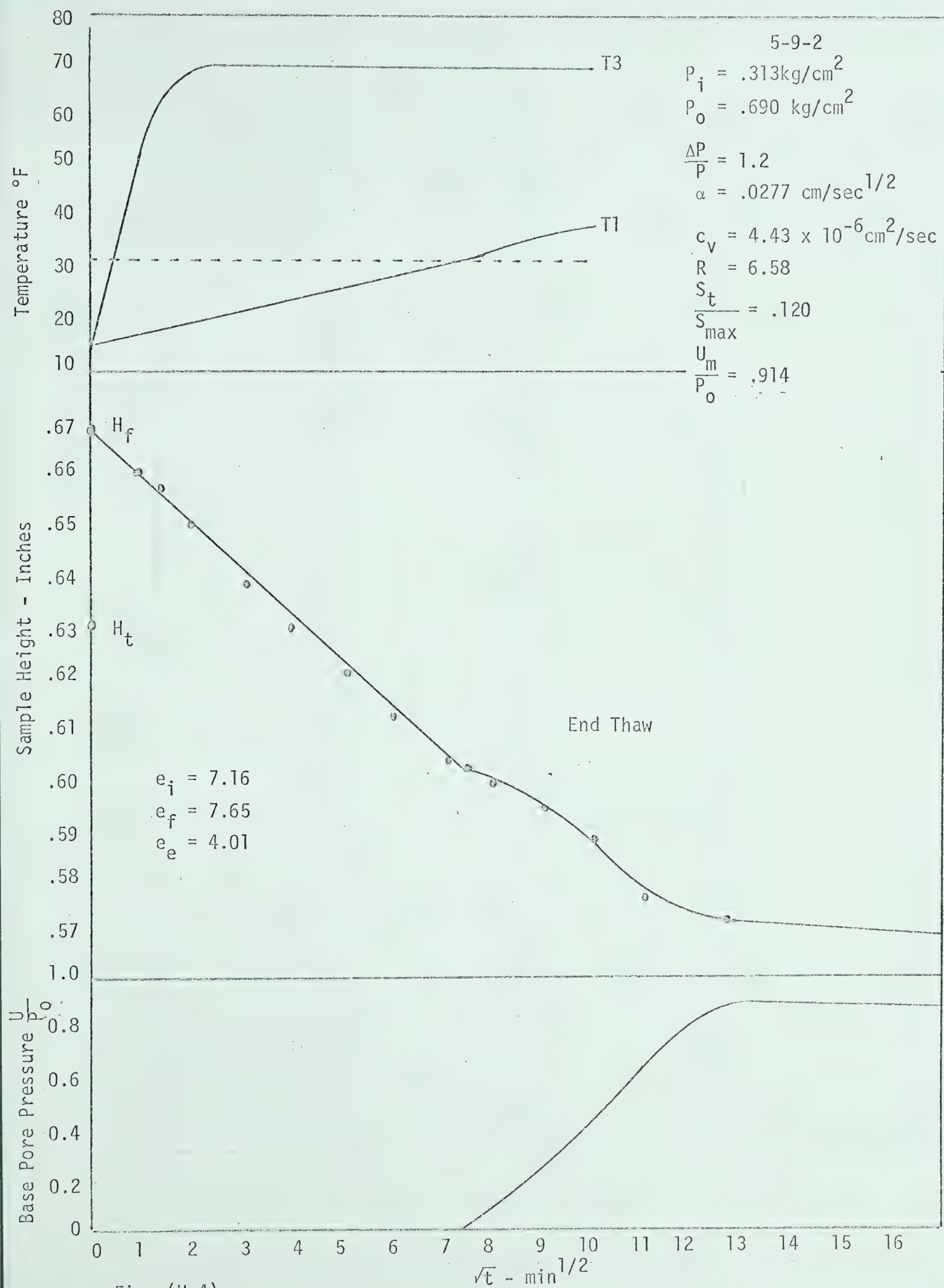


Fig. (H.3) Post Thaw Settlement and Pore Pressures Test 5-9-1



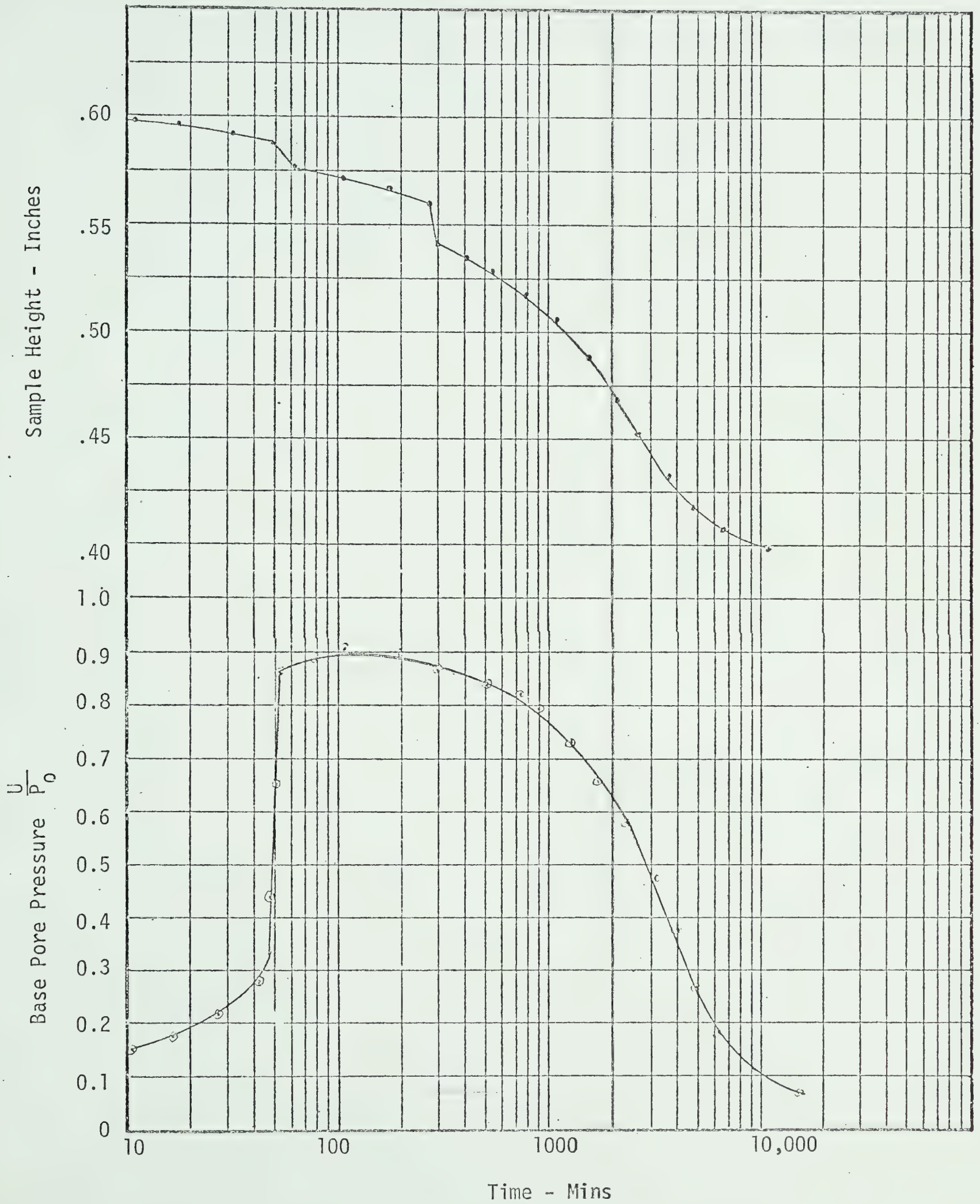


Fig. (H.5) Post Thaw Settlement and Pore Pressures - Test 5-9-2

B30016